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Public Roads

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Volume 34

April 1966-February 1968

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

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Night scene showing Brent Spence Bridge between Cincinnati, Ohio, and Covington, Ky., on Interstate 75.

315229

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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NOTICE

It has been called to our attention that some copies of the February 1966 magazine, "Public Roads, A Journal of Highway Research," were received in a damaged condition and/or were missing pages 249 through 254 and pages 271 through 276. If you were unfortunate enough to receive such a copy, please notify the editor—if possible return the damaged copy—and a replacement will be made.

Errata

Please correct the second listing in the February 1966 issue of this magazine under "IN THIS ISSUE" to read:

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Contents of this publication may be reprinted. Mention of source is requested.

Traffic Flow Responses to Unannounced Increases in Progression Speeds of Signal Systems

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

by[1] **RICHARD D. DESROSIERS, Highway Research Engineer,**
and CLYDE H. LEIGHTY, Highway Engineer Trainee,
Traffic Systems Division

To determine the effect on vehicular traffic when a change is made in the speed of progression of progressively timed signal systems, the time required for responses to the changes must also be determined. This knowledge will be helpful in designing traffic responsive signal systems. This article is a report on a study made of the time required by drivers to adapt to a change in speed of progression from 27 to 33 m.p.h. and from 33 to 40 m.p.h. on a 1-way urban arterial when the drivers had no knowledge of the change. The authors believe that better adaption could be obtained if information on the changes in the signal system were available to the drivers.

For the three speeds of progression studied, the drivers had neither completely adapted to the change in progression speed from 27 to 33 m.p.h. after a period of 2 months nor to a change from 33 to 40 m.p.h. after a period of approximately 1 month. The authors also believe that large traffic volumes are a limiting factor in the ability of drivers to adapt to a change in progression—choice of speed is restricted by the speed of other vehicles in the traffic stream. This belief was confirmed by the greater adaptation that was made for both changes in progression speed when the volume of traffic was lighter than during the peak rush hours.

Introduction

THE FIRST phase in a continuing study of the time required by drivers to adapt their driving pattern to changes in traffic control signal systems is discussed in this article. These studies are being conducted by the Bureau of Public Roads to gain an insight into the ability of drivers to adapt to changes in speeds of progression and other changes in the signal system, and the resultant effect on vehicular traffic. Determination of the time it takes drivers to adapt to these changes in progression of the timing of signals, and other modifications made or contemplated by traffic engineers, affects the evaluation of any changes. In the first phase, an attempt was made to determine the time required by drivers to adapt to changes in the progression of a coordinated signal system on 13th Street NW., Washington, D.C.—a 1-way urban arterial street—when no information regarding the change had been provided. Any knowledge the drivers had of the change had to be obtained from the experience of driving through the test section. Much of the field data gathered in the first phase of the study are expected to be used during the other phases.

Three speeds of progression were studied: 27 m.p.h., the initial condition; 33 m.p.h.; and 40 m.p.h. Data to establish the existing traffic characteristics for the initial condition of 27 m.p.h. were collected early in February 1964. The progression speed was changed on February 21, 1964 to 33 m.p.h.; data collection was started immediately and continued until the pattern of driver adaptation to the change in speed of progression had been clearly established. The speed of progression was changed April 17, 1964 to 40 m.p.h., and data collection was continued until completion of the first phase of the study, May 23, 1964.

Conclusion

On the basis of data collected in the study reported here, the conclusion has been made that a considerable length of time is required for drivers to adapt to changes in the speed of progression of a progressively timed signal system on an urban arterial street, particularly when the driver has no knowledge of this change. The drivers did not adapt in a day, a week, or even a month. Drivers had not fully adapted to a change in progression speed from 27 to 33 m.p.h. after 2 months nor to a change from 33 to 40 m.p.h. after approximately 1 month. The drivers made a slight adaptation to the change during the peak hour of traffic but seemed to adapt more readily to the changes when traffic was lighter at the off-peak hours.

The authors also recommend that research be undertaken in the future to determine the adaptation time of drivers to changes in the speed of progression of the signal system when the drivers are provided information of the change by signs or other media.

Test Site

Thirteenth Street, an arterial street in Washington, D.C., with a progressively timed signal system was used for the study site. As shown in figure 1, the limits of the study site extended from Park Road to Euclid Street, approximately 2,700 feet. Thirteenth Street is a 4-lane arterial on which traffic is 1-way southbound from 7 to 9:30 a.m.; 1-way northbound from 4 to 6:30 p.m.; and 2-way for the rest of the day. Data reported here were collected only during the morning rush period when all traffic was southbound in each of the 4 lanes. The existing signals were coordinated for 27 m.p.h. speed of progression. An 80-second cycle and a through band for traffic of 55 seconds were in effect.

Total turning movements from the test site constituted 18 percent of the average hourly traffic volume; these turns were made as follows from 13th Street to: Columbia Road, 5.3 percent; Harvard Street, 3.3 percent; Irving Street, 3.0 percent; and other, 6.4 percent. Vehicles turning onto 13th Street within the test site constituted 16.5 percent of the average hourly traffic, but these vehicles did not cause any interference with the study platoon because of the separation by traffic signals.

Study platoon

The term study platoon as used in this article refers to the group of vehicles that entered the test site on a green phase of the signals southbound on 13th Street. As the

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[1] This study was made with the cooperation of the District of Columbia Department of Highways and Traffic. Special acknowledgment is made for the assistance of Joseph Rice, Chief of the Traffic Operations Division, and Mike Flanakin, Head of the Research Section, in obtaining a suitable test site and for their continued cooperation throughout the study.

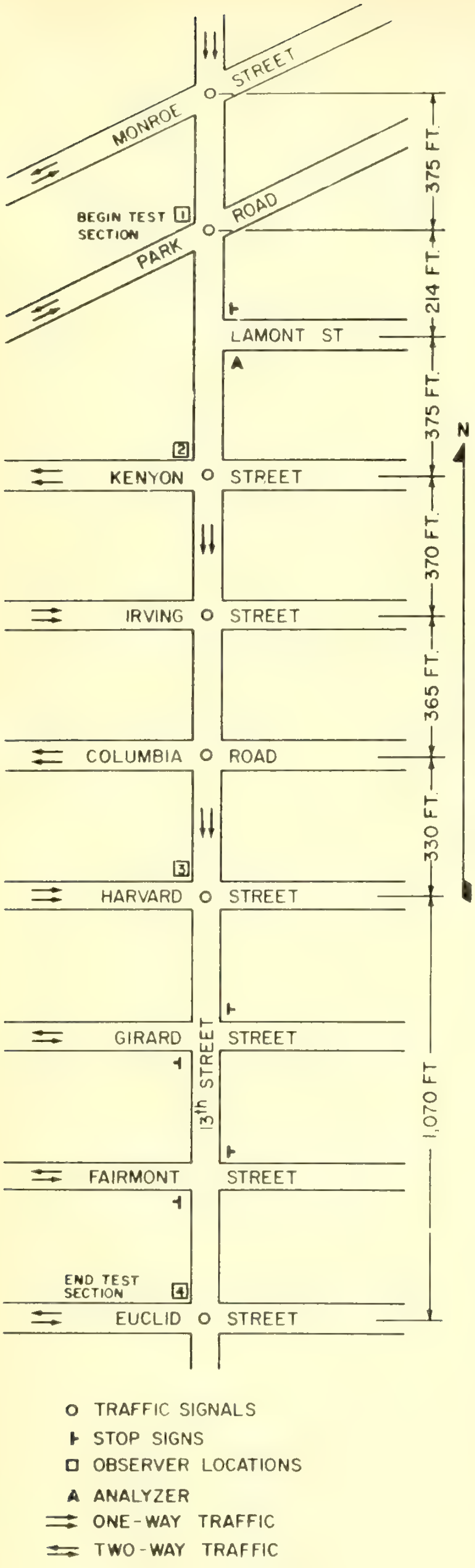

Figure 1.—Test site.

group of vehicles proceeded through the test site, some vehicles turned off or onto 13th Street. Also, others were stopped at signalized intersections within the test site. Thus the study platoon was not a fixed group of vehicles but a flexible group, which reflected the effect of turns and stops on the composition of the group that entered the test site. Traffic during the study period had essentially the same composition: passenger cars, 97+ percent; buses, 1.0 percent; and trucks, 1.0 to 2.0 percent.

Observers were stationed along 13th Street at the intersections with Park Road, Kenyon, Harvard, and Euclid Streets, as shown in figure 1. Each observer had a portable radio and five watches mounted on a clip board; the watches were adjusted so that each observer could start the five simultaneously but stop them individually. The observers recorded the speed, volume, and headway characteristics of the study platoon as it moved through the test site. To obtain usable data, accurate identification of a specific study platoon by each observer was essential. Platoon identification was difficult because different vehicles alternately assumed the lead position. To assist the observer in identifying each study platoon, radio communication from a spotter car in each platoon was used.

The spotter car assumed a position at or near the beginning of the formation of the study platoon at Monroe Street, a street preceding the beginning of the test site. On the amber signal, the radio operator in the spotter car transmitted an alert message to all observers. Then a start signal was given and each observer started his five watches. This start signal coincided with the beginning of the green phase at Monroe Street. As the study platoon moved down the street, the driver of the spotter car maintained a position within the platoon so that he could always see the lead vehicle. As the platoon approached the location of each observer, the radio operator notified him of the color, make, and year of the lead vehicle and the lane in which it was traveling so that each observer could identify the beginning of each study platoon.

As the study platoon passed each observer, he stopped the watches in sequence as the 1st, 10th, 20th, 35th, and the 50th—or last vehicle if less than 50—in the platoon passed. After the passage of time at which each of these vehicles had passed the observer was recorded, the watches were reset at zero for the timing of the next platoon. The traveltime southbound seldom exceeded 2 minutes, but the return trip of a spotter car frequently took from 8 to 10 minutes on parallel streets. Therefore, two spotter cars were used so that it was possible to study a platoon at intervals of approximately 6 minutes.

Travel speed

The recorder in the spotter car recorded the total traveltime through the test site, the time lost for each delay, and the cause of the delay. This method was similar to the floating car technique except for the instruction to the driver to maintain a position near the lead vehicle in the platoon. Thus, a running speed rather than an average speed was obtained because the lead vehicle at the front of the platoon was not stopped within the test site. This technique also was used to collect travel speed data on days when the observers and the traffic analyzer were not used. This permitted periodic observation of the speed trend and required only a few field personnel.

Traffic analyzer

The Bureau of Public Roads traffic analyzer was located just south of the Lamont Street intersection within the test site. Speed, volume, and headway data were collected by individual lanes for all traffic during the study period. By use of a coding system, each vehicle was classified as to whether it came from above Monroe Street or from off Monroe Street, Park Road, or Lamont Street. Data from the analyzer were used for speed, volume, and headway distributions and for comparison with data provided by the observers.

Data Analysis

An analysis of variance was performed to determine statistically the significance of any difference in the data reported by the observers for several volume groupings. Data for speed and headway of the study platoons were analyzed in relation to the positions of the four observers and vehicle position within the platoon. Statistically significant differences were subjected to a simple Fisher (F) test. When no statistically significant differences in variances were obtained, it was concluded that the observed difference in the analysis of variance was the result of a significant difference in means. The 5-percent level of significance was used for all tests. A 5-percent level of significance means that even without significant statistical differences, an actual difference as large as that obtained could occur by chance alone 5 times in 100. Only significant variables are discussed in this article. A regression analysis was performed on the speed data obtained from the spotter vehicle. More information on the regression analysis is presented later.

Traffic Characteristics for 27 M.P.H. Progression Speed

The daily traffic volume on the test site during 1963 on Tuesdays, Wednesdays, and Thursdays was fairly constant according to traffic counts made by the District of Columbia Department of Highways and Traffic. The mean daily volume of traffic was approximately 26,700 vehicles and never varied more than 5 or 6 percent. Also, very little variation occurred between weeks or month; this variation being ±5 percent of the mean reflected by the daily variation. The mean of the peak hour volume was 3,465 vehicles and the fluctuation was ±5 percent of the peak hourly volume. On the basis of past volume trends, it was determined that a representative sample of normal traffic could be obtained by collecting data on any Tuesday, Wednesday, or Thursday. Data on traffic volume collected in early February 1964 are reported in figure 2. Volume is shown for each 12-minute period from 7:18 to 9:30 a.m. The peak 12-minute volume of 710 vehicles occurred during the period from 8:06 to 8:18 a.m. The traffic volumes recorded in February 1964 were comparable to those reported for 1963.

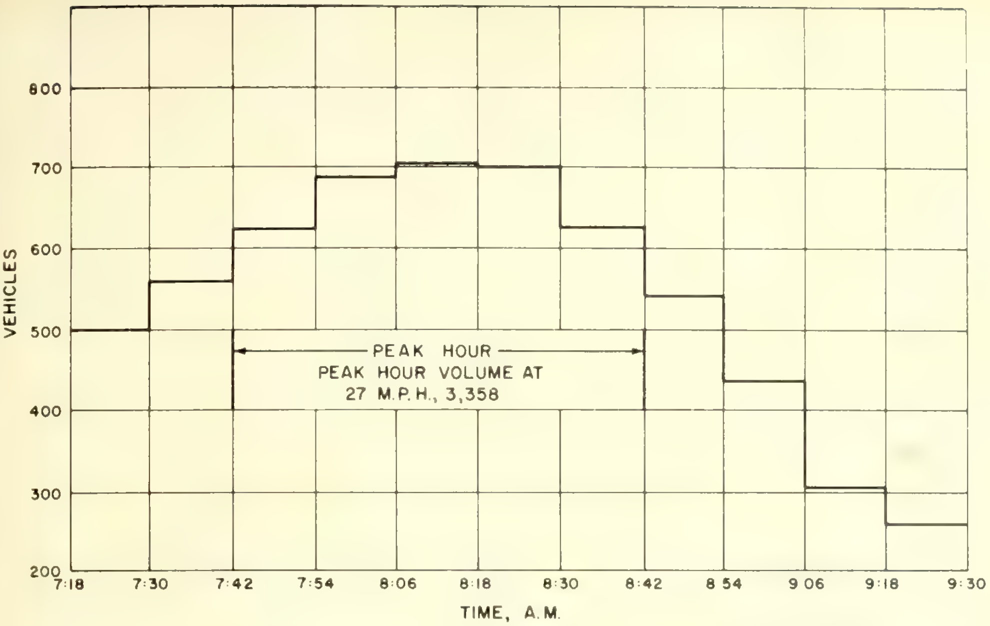

Figure 2.—Average traffic volume for all lanes for 12-minute periods for 27 m.p.h. speed of progression.

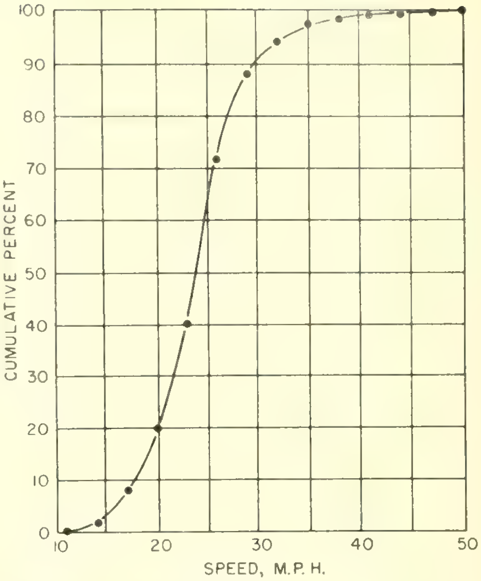

Figure 4.—Distribution of spot speeds for all lanes at 27 m.p.h. speed of progression.

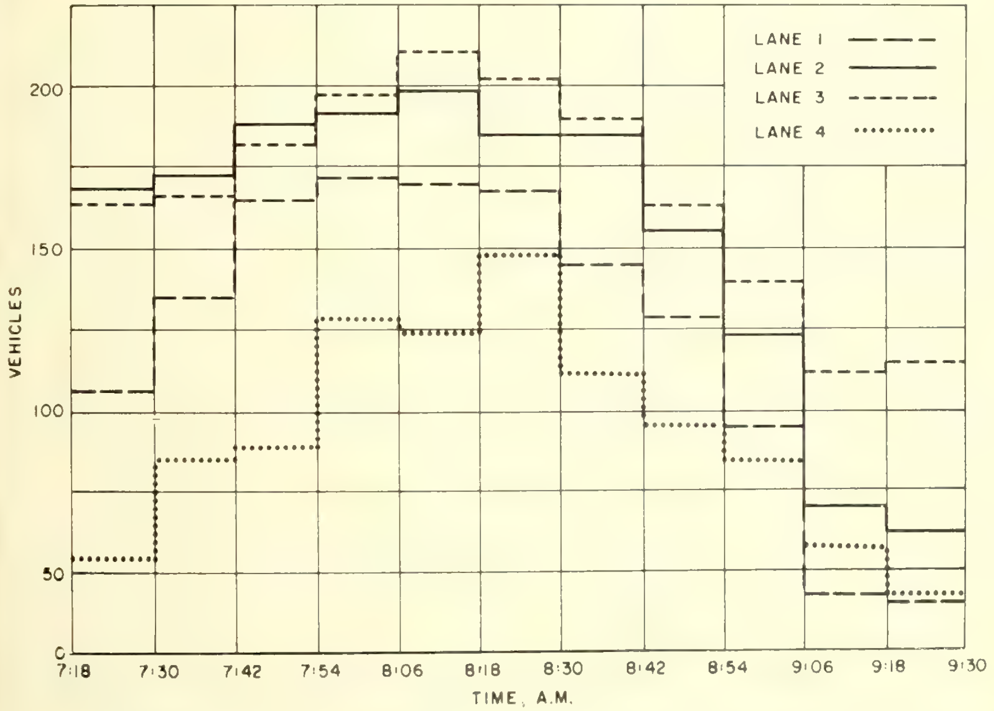

Figure 3.—Average traffic volume per lane by time increment for 27 m.p.h. speed of progression.

The traffic volume by lane is reported in figure 3. Each of the center lanes carried considerably more vehicles, in all time periods, than the two curb lanes. Traffic in the right curb lane was especially low for all time periods studied. Even in the peak hour this lane carried only about one-half to two-thirds the traffic volume carried by the center lanes. Drivers in the right curb lane are restricted by turning vehicles and buses; and, generally, drivers not wishing to turn avoid this lane. This reasoning is confirmed by the large percentage of through traffic in relation to turning traffic noted in the study reported here.

Speed

The distribution of spot speeds obtained with the traffic analyzer, which was placed midway between the signalized intersections of Park Road and Kenyon Street, is shown in figure 4. The speeds of all vehicles observed during the period from 7:30 to 9:30 a.m. for a typical study day are included. The mean speed was 23.9 m.p.h. with a standard deviation of 5 m.p.h. With this mean and standard deviation, 67 percent of all vehicles can be expected to travel at speeds within the range from 18.9 to 28.9 m.p.h.; and 95 percent can be expected to travel at speeds from 13.9 to 33.9 m.p.h. The 85th percentile speed, a speed often used by traffic engineers to establish speed limits, was 28.4 m.p.h. The distribution of speeds by lane is shown in table 1. The speed in lane 1—the far left lane when travel is southbound—was significantly lower statistically than in the other three lanes, in which speeds were nearly identical. The differences in mean speed, 85th percentile speed, and standard deviation for the different lanes were small.

Headways

Times recorded for the passing of the 1st, 10th, 20th, 35th, and 50th vehicles were used to compute the average time headway per vehicle per lane for each of the several groups of vehicles by the following formula: Headway is defined as the time between successive vehicles.

$$H=\frac{TL}{N-1}$$

Where,

H= Average headway in seconds per vehicle per lane for the group of vehicles being considered.

T= Passage time for the group in seconds.

L= All four traffic lanes.

N= Number of vehicles in group (10 or 15).

Headway was computed in this manner for groups of vehicles of: 1 through 10, 11 through 20, 21 through 35, and 36 through 50. As no significant differences in headway were observed between these groups, the data were combined for further analyses. This does not mean that no differences occurred: it does mean that if differences existed they could not be detected by this technique. Figure 5 shows the average time headway in seconds per vehicle for the time of day for the platoons studied during the 27 m.p.h. speed of progression. The range of average headways was from 2.5 to 4.0 seconds. The headway for the mean peak hour platoon was 2.7 seconds. T

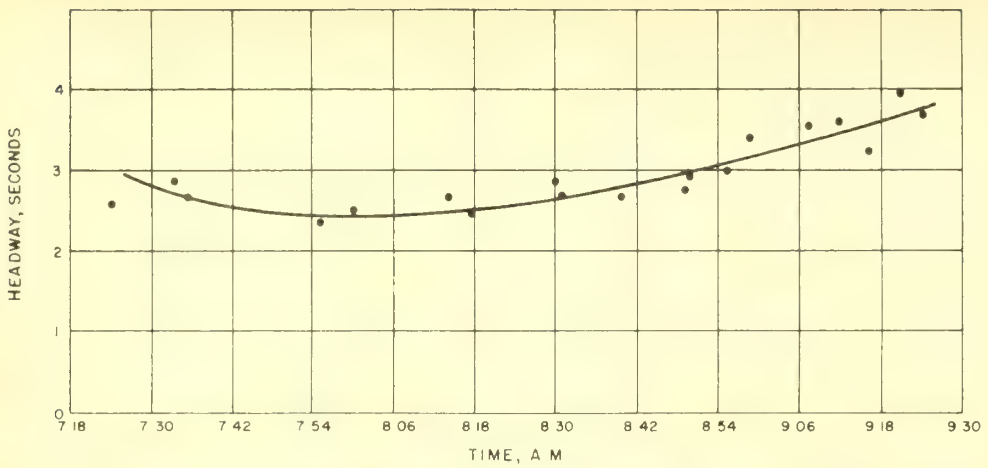

Figure 5.—Average time headways related to time of day for platoons during 27 m.p.h. speed of progression.

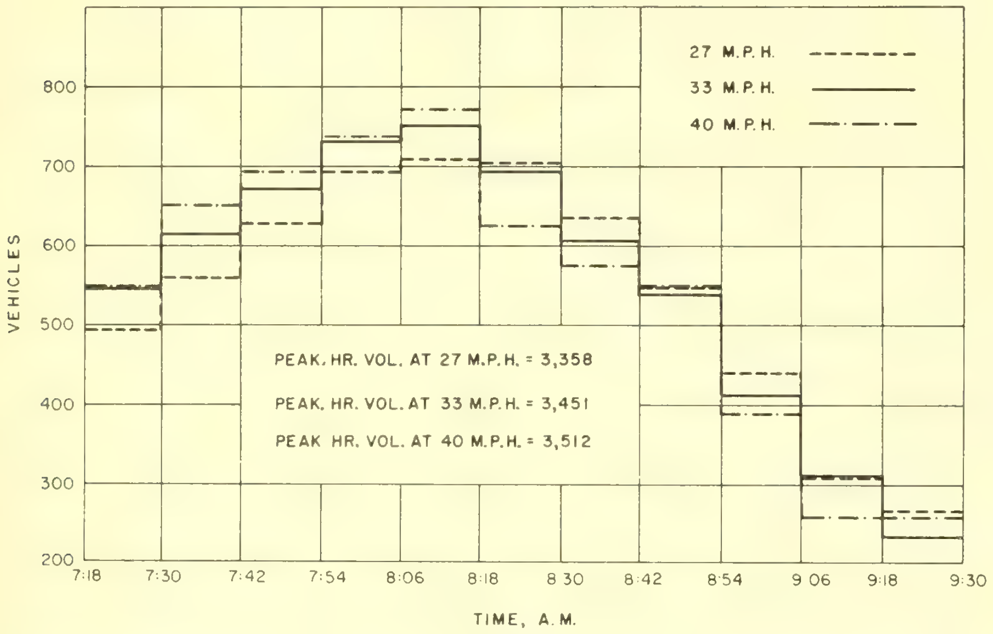

Figure 6.—Average traffic volume for all lanes by time increment for three speeds of progression.

Table 1.—Speed characteristics analysis

Lane	Mean speed	Standard deviation	85th percentile speed
1	23.0	4.74	27.5
2	24.6	5.25	29.0
3	24.0	5.01	28.2
4	24.5	5.67	29.3

Table 2.—Mean, standard deviation, and 85th percentile spot speeds by lane for each speed of progression

Speed of progression, m.p.h.	Mean, $\overline{X}$	Standard deviation, S_x	85th percentile
Lane 1:			
27	23.0	4 74	27.5
33	26.3	4.35	30.8
40	26.7	5.01	29.8
Lane 2:			
27	24.6	5.25	29.0
33	27.5	3.24	32.5
40	29.2	5.39	32.7
Lane 3:			
27	24.0	5.01	28.2
33	26.0	4.53	30.5
40	27.0	5.40	31.0
Lane 4:			
27	24 5	5.67	29.3
33	26.1	5.49	31.5
40	26.5	5.30	30.5

headway for the mean off-peak hour platoon was 3.5 seconds.

Effect of Increase in Speed of Progression

Traffic volume conditions for typical days at each speed of progression are shown in figure 6. The peak hour volumes of traffic increased slightly, being 3,358 at 27 m.p.h., 3,451 at 33 m.p.h., and 3,512 at 40 m.p.h. It was not possible to establish whether the increase that accompanied the change in speed was a true increase or a chance occurrence because the increases were within 5 percent of the historical daily mean. It seems that as the speed of progression was increased, a higher volume per 12 minute period occurred until the peak hour volume was attained; but when the peak hour volume was reached the trend was reversed and traffic volume per 12-minute period was less.

Speed Distribution

The distribution of spot speeds for all vehicles passing the traffic analyzer during the study period the last study day for each speed of progression is shown in figure 7. The mean spot speed increased from 23.9 m.p.h. to 26.5 m.p.h. and then to 27.9 m.p.h. respectively for the 33 m.p.h. and 40 m.p.h. progression speeds. Also, the variance was essentially unchanged when the speed increased from that of the original condition. The 85th percentile speed increased from 28.4 m.p.h. to 31.3 m.p.h., when the speed of progression was increased from 27 to 33 m.p.h. but little change occurred when the speed of progression was increased to 40 m.p.h.

The mean speeds, standard deviations, and 85th percentile speeds for each lane are listed in table 2. Lane 2 had the largest increase in speed (4.6 m.p.h.) and lane 4 had the least (2.0 m.p.h.). In lane 1—where speed was significantly slower at the 27 m.p.h. speed of progression than in the other lanes—no lag in speed was noted at 33 m.p.h. and 40 m.p.h. speeds of progression. There was no significant change in the standard deviation. The 85th percentile speed also increased with the increase in progression speed. Most of the increase in mean and 85th percentile speed can be accounted for by the increase from 27 m.p.h. to 33 m.p.h., with little additional increase noted when the progression speed was increased to 40 m.p.h.

The mean and 85th percentile speeds may mask the true magnitude of the adaptation of drivers when traffic volumes are smaller because a large proportion of the observations were recorded in the peak hour when the driver's choice of speed was limited by traffic volume considerations,this is shown in figure 8. For each of the speeds of progression, the spot speed decreased as the volume increased. Of greater importance, however, is the apparent converging of the curves as the volume increased, which indicates that greater adaptation was being made at the smaller traffic volumes.

Speed of Lead Vehicle

Any adaptation to change in speed of progression should be reflected by the lead vehicle in the platoon as this vehicle is free of interference from vehicles in front of it. However, if the speed of the lead vehicle is to be used as an indicator of driver adaptation, this speed must be established as representative of the entire platoon. In table 3 the running speed of the lead vehicle is compared

(Continued on p. 13.)

Variability in an Asphalt Concrete Mix

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported[1] by EDWARD R. OGLIO and
JOSEPH A. ZENEWITZ, Chemists,
Materials Division

The research reported in this article was conducted as part of the first phase in a broad program being undertaken by the Bureau of Public Roads to develop control and acceptance procedures, based on statistical quality control techniques, to all aspects of highway materials and construction. As the program is now constituted, the first phase calls for determination of two basic statistical parameters—the average and the standard deviation—for the materials and structural elements now being used in good highway construction.

In the work reported here, averages and variations in temperature, asphalt content, and aggregate gradation were determined in an asphalt concrete wearing course mix produced for a construction job. A statistical analysis was made to show the effect of test method, sampling procedure, and material (batch-to-batch) variation on the overall variations obtained.

Introduction

THE CONSTRUCTION of quality highways and the procedures to assure achievement of quality have always been primary concerns of those involved in the many phases of planning, designing, and building highways. In recent years highway programs of increased magnitude to accommodate larger traffic volumes, faster speeds, and heavier wheel loads have demanded increasingly higher levels of quality in highway construction and improved control and acceptance procedures to assure attainment of such levels.

The dictionary gives more than 15 definitions for the word quality. However, a definition that perhaps applies best to highway construction describes quality as the degree to which a product satisfies a consumer. In the highway industry a product could be the finished highway, any or all of its structural components, and any of the constituent materials used in construction of the finished product or of its components. A consumer in this context would be a State highway department or other government department, a contractor, a subcontractor, or even a private individual.

Findings Summarized

As the study discussed in this article covered but one type of asphalt mixture produced at one plant for a single construction project, no general conclusions can be drawn from the data at this time. Such conclusions should await acquisition of results from similar studies contemplated under the nationwide Bureau of Public Roads Quality Control Program. Pending acquisition of these additional data, the significant results obtained are summarized as follows:

- Inherent process variability was the major factor contributing to variation in mixture composition, as determined by extraction and gradation tests. When variability is expressed as variance, approximately 60 percent of the variation in asphalt content and 85 to 98 percent of the variations in gradation were attributable to the plant process. Variation attributable to the sampling and test procedures accounted for the remainder of variation in composition.
- The sampling and testing procedures used were minor factors in the variations observed in mix composition; they accounted for substantially less than half of the variation for each item of composition.
- Deviations of the test averages from the design values were the major cause of the deviation of individual test results from the permissible limits (tolerances).
- The feasibility of conducting studies of this type without appreciable interference with the normal course of plant production was confirmed.

The Concept of Quality

Two aspects (*1*)[2] are involved in the concept of quality: (1) quality of design and (2) quality of conformance. Quality of design is usually incorporated in plans, specifications, special provisions, or other formal documents, any or all of which describe the item wanted or produced. It is the levels of quality of products that are intended to perform the same basic function. For example, a box camera and a 35-mm. camera serve the same basic function. However, they differ substantially as to the levels of design incorporated in each. Similarly, highways are designed at different levels of quality depending on the expected traffic, climate, and other considerations. Obviously a highway or pavement whose function is to bear high-density, high-speed, heavy wheel load traffic is designed at a higher level and requires better materials and material control than one intended to serve low-density, low-speed, and lighter wheel load traffic.

Quality of conformance, however, refers to the fidelity or degree to which a product conforms to its design. A high degree of conformance to the design elements means a high degree of assurance that the designed quality has been attained. A 35-mm. camera that does not take pictures differs as to quality of conformance from one that does, to use an extreme example. Usually, a variation will exist in the ability of individuals using 35-mm. cameras that had been made to the same design and specifications or even in individual cameras from the same production line. Such variation is a manifestation of the fact that variation is a universal phenomenon whether products, processes, measurements, or materials are concerned. The better the control of such variations, the closer the individual items will conform to the design quality level under which they were produced. In production, allowance is made for variation by permitting design tolerances in the specifications under which the items are produced.

Although present knowledge and capabilities permit attainment of almost unlimited levels of quality in both design and conformance, the limiting factor of cost must always be considered. The higher these levels, the higher will be the cost of achieving them. However, there are levels at which the costs of quality achievement are in balance with the value for the consumer or, at least, in balance with what he needs and with what he can pay. In the highway field these levels should be the desired goals.

Bureau of Public Roads Program

Different methods have been used to achieve quality goals. Those based on statistical concepts have been widely and successfully used in industry. Such methods are generally referred to as quality control methods when used by producers and as acceptance plans when used by purchasers or consumers. However except for some notable exceptions such as construction of the Garden State Parkway in New Jersey, the Illinois toll road, and t

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[1] Presented at the annual meeting of the Association of Asphalt Paving Technologists, Philadelphia, Pa., Feb. 15–17, 1965.

[2] Italic numbers in parentheses indicate the references listed on page 11.

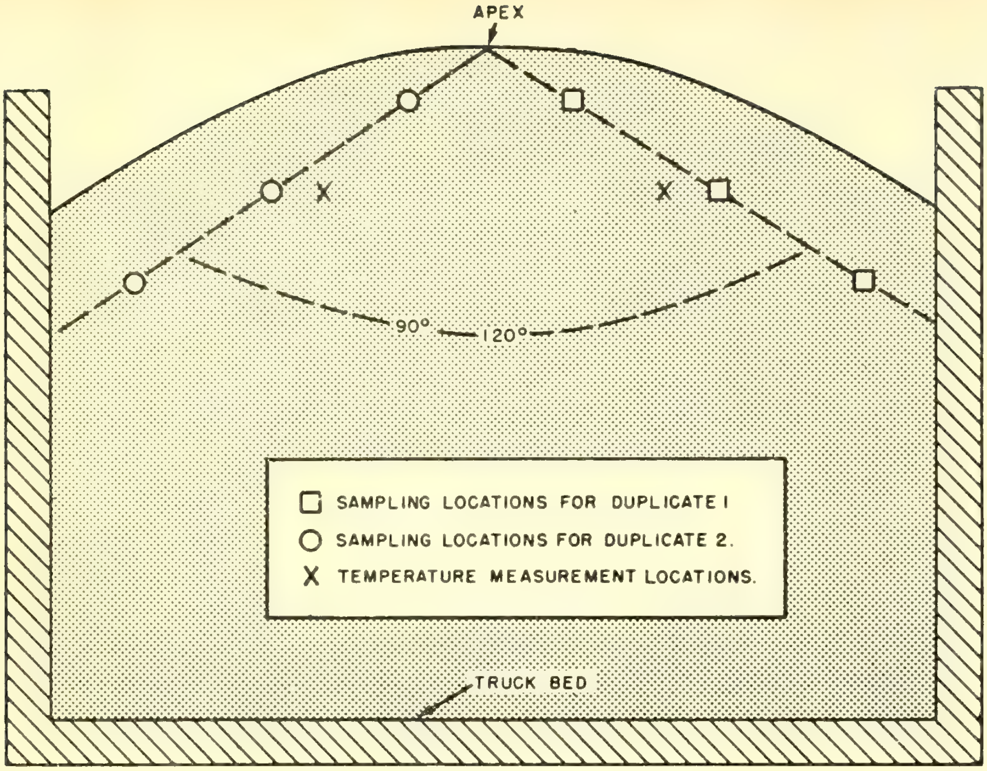
APEX
90°
120°
☐ SAMPLING LOCATIONS FOR DUPLICATE 1
○ SAMPLING LOCATIONS FOR DUPLICATE 2.
X TEMPERATURE MEASUREMENT LOCATIONS.
TRUCK BED

Figure 1.—Schematic of sampling locations.

AASHO Test Road, quality control procedures have not been used to the fullest by the highway industry.

Accordingly, the Bureau of Public Roads has undertaken a broad program aimed at applying statistical techniques to the development of improved procedures for control and acceptance in all phases of highway construction. Basically the objective of the program is to develop procedures that will assure, with a known degree of confidence, production and acceptance of the materials and structural elements of highways at the levels of quality required by modern traffic conditions and economic considerations. Attainment of this objective, in three more or less consecutive phases, is being sought through coordinated efforts with the States and other segments of the highway industry. These efforts are explained in the following paragraphs.

- The first phase is aimed toward determination of the basic statistical parameters associated with the significant requirements and characteristics specified or designed for highway materials and structural elements through studies of a broad spectrum of current good highway design and construction. The basic parameters required for the overall program, as now constituted are the average and the variability of the measurable characteristics and requirements.
- The second phase includes development of guidelines, based on statistical concepts, for preparing highway specifications; together with development of sampling and acceptance plans for evaluating conformance to the specifications at known levels of confidence.
- The third phase includes development of acceptable levels of quality and uniformity (tolerances) that will provide an optimum balance between construction, performance, and maintenance, and overall cost.

Purpose of Study

The study discussed in this article was undertaken primarily to obtain some of the data required to determine the statistical

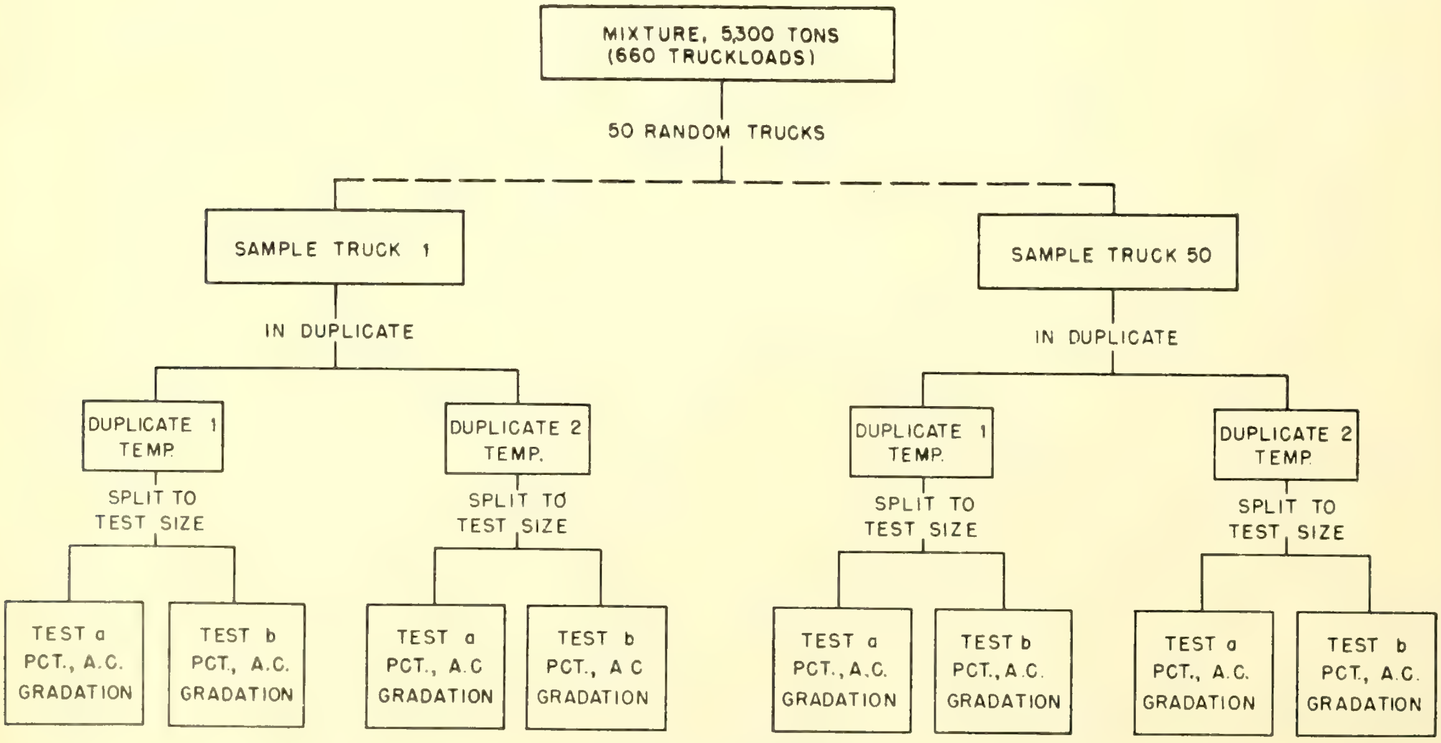
MIXTURE, 5,300 TONS (660 TRUCKLOADS)
50 RANDOM TRUCKS
SAMPLE TRUCK 1
SAMPLE TRUCK 50
IN DUPLICATE
IN DUPLICATE
DUPLICATE 1 TEMP.
DUPLICATE 2 TEMP.
DUPLICATE 1 TEMP.
DUPLICATE 2 TEMP.
SPLIT TO TEST SIZE
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Figure 2.—Sampling plan.

Table 1.—Summary of extraction test results

Test property	Mix design (JMF)	Number of tests	Test results				
			Average	Standard deviation	Percent of tests—		
					Within JMF	Below JMF	Above JMF
Mix temperature ° F	305 ±20	100	305.5	[1] 18.0	70.0	[1] 18.0	[1] 12.0
Asphalt content percent	6.0± 0.3	200	5.955	0.120	100.0	0.0	0.0
Gradation, aggregate passing sieve:							
½-inch percent	100	200	100.0		100.0	0.0	0.0
⅜-inch do	93 ± 7	200	97.5	1.72	100.0	0.0	
No. 4 do	63 ± 7	200	62.0	4.18	94.0	[2] 4.5	[2] 1.5
No. 10 do	42 ± 4	200	38.9	2.13	74.5	[3] 25.5	0.0
No. 20 do		200	26.4	2.05			
No. 40 do	18 ± 4	200	18.3	1.69	100.0	0.0	0.0
No. 80 do	7 ± 4	200	9.5	1.46	89.5	0.0	[4] 10.5
No. 200 do	4 ± 2	200	5.59	1.16	77.5	0.0	[4] 22.5

[1] Ascribed mainly to changes in plant operation made to compensate for changes in atmospheric temperatures.
[2] Ascribed mainly to high variability.
[3] Ascribed mainly to average lying below design value.
[4] Ascribed mainly to average lying above design value caused at least in part by dry sieving of hot bin aggregates during plant control.

Table 2.—Summary of combined hot bin gradations test results

Aggregate passing sieve, percent	Mix design (JMF)	Test results[1]—	
		Average	Standard deviation
½-inch	100	100	
⅜-inch	93±7	98.8	0.15
No. 4	63±7	65.2	2.14
No. 10	42±4	41.7	1.41
No. 40	18±4	16.9	1.13
No. 80	7±4	7.04	1.32
No. 200	4±2	3.75	1.11

[1] Ten tests were made for each size of aggregate.

parameters under phase 1 of this quality control program. Secondarily, the study was made to evaluate some of the factors contributing to variability in the characteristics studied as a guide for future efforts to improve uniformity and control.

The work discussed in this article concerned: (1) Determinations of the averages and the variations in temperature, asphalt content, and aggregate gradation in an asphalt concrete wearing course mixture produced by one plant for a normal job; and (2) an evaluation of the effects of inherent process variability, sampling procedure, and test method (measurement process) on the variations obtained.

Methods and Materials

The job selected for this study was a resurfacing of an existing pavement requiring an estimated 7,700 tons of asphalt concrete for the wearing course. However, for practical reasons, only the last 5,300 tons of production were sampled to obtain the data reported for this study. The mixture consisted of a coarse aggregate of crushed trap rock, river sand, and screenings combined with an 85–100 penetration grade asphalt cement for a job mix formula (JMF) of:

Aggregate, sieve size	*Percent*
½-in.	100
⅜-in.	93±7
No. 4	63±7
No. 10	42±4
No. 40	18±4
No. 80	7±4
No. 200	4±2
Asphalt cement	
Mix basis	6±0.3

Plant and Production

The mix was produced in 2⅔-ton batches in a batch plant that had a 3-ton capacity, twin-shaft pugmill mixer. Heated aggregates were stored in three hot bins that discharged directly into the weigh-hopper. The proportions of hot bin aggregates and asphalt flowing into the mixer were manually controlled. The mixing cycle used for the total production was a 10-second dry mixing of the aggregates followed by 30 seconds of wet mixing. A temperature of 305° F. ±20° F. was specified as mixture temperature. However, the job ran into late November, and increases in mixture temperature were ordered on days when atmospheric temperatures dropped below 50° F. Normal plant control practices were used by plant and inspection personnel throughout the entire production. These practices included, among others, day-to-day control of aggregate gradation through daily hot bin gradation tests made by the inspector, who performed his control activities independently of the study.

In making the hot bin gradation tests, samples were taken from the aggregate stream directly under each bin and dry sieved manually. The sieve sizes used were as follows: (1) Coarse bin: ½-in., ⅜-in., and No. 4; (2) medium bin: ⅜-in., No. 4, and No. 10; and (3) fine bin: No. 4, No. 10, No. 40, No. 80, and No. 200. The combined hot bins gradation was obtained by calculations from the gradation test results and the respective bin proportions of 8, 40, and 52.

Sampling

The mixture selected for the study totaled 5,300 tons and was transported in 660 truckloads. Samples were taken from 50 randomly selected trucks. To ensure randomization a table of random numbers was used in selecting the individual trucks to be sampled. Duplicate samples were taken from each truck. A duplicate sample consisted of a composite of three scoops of mixture, each scoop containing approximately 3 pounds. These scoops of mixture were taken from the pile in the truck at a depth of approximately 1 foot and at three levels along an imaginary straight line from the apex to the bottom. Duplicate samples were taken from different lateral locations in the pile, 90 degrees to 120 degrees apart as measured from the apex. A schematic representation of this procedure is shown in figure 1. One temperature measurement was made at each duplicate sample location (fig. 1). Each duplicate sample was split by a sample splitter a sufficient number of times to provide two samples of approximately 1,200 grams each for extraction and gradation tests.

A schematic representation of the sampling plan is shown in figure 2. Use of this plan permits an evaluation through an analysis of variance of the effects of inherent process variability, sampling, and method of test on total variability.

Test Methods

Asphalt content was determined by ASTM Method D 2172–63T Method A, using benzene as the solvent (*2*). No correction was made for moisture content because spot checks had indicated that moisture was negligible. AASHO Designation: T 30–55 (*3*) was used for sieve analysis of the extracted aggregates. Temperatures were measured at approximately 8-inch depths at the two temperature test sites in the mixture pile, as shown in figure 1. These measurements were made with calibrated dial thermometers immediately after the last batch of mix had been dumped into the truck.

Analysis of Variance

The analysis of variance to evaluate the factors contributing to variability was based on the following relation:

$$(\sigma')^2 = (\sigma_a)^2 + (\sigma_s)^2 + (\sigma_t)^2$$

Where:
$(\sigma')^2$ = Overall variance.
$(\sigma_a)^2$ = Inherent process variance.
$(\sigma_s)^2$ = Sampling variance.
$(\sigma_t)^2$ = Variance caused by test method.

Details of the method used to compute the variance (*4*) are given at the end of this article.

Results

A summary of the extraction test results showing averages, standard deviations, and percentages within or outside the ranges permitted by the job mix formula for each measured property is given in table 1. Similar results for the combined hot bin gradations are given in table 2. These tests were made

Table 3.—Extraction test results

Sample			Mix temperature	Aggregate passing sieve—							A.C. mix basis
Number	Duplicate	Test		⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200	
			°F.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
1	D₁	a	304	98.7	63.4	38.8	24.7	14.9	6.4	3.51	5.75
	D₁	b		97.8	62.9	38.2	24.1	14.8	6.5	3.53	5.69
	D₂	a	303	98.0	65.2	38.9	24.2	14.8	6.5	3.64	5.76
	D₂	b		98.5	63.1	38.4	24.1	14.8	6.3	3.52	5.74
2	D₁	a	308	98.8	60.8	38.2	25.1	16.0	7.7	4.48	5.73
	D₁	b		98.3	61.4	38.7	25.3	16.1	7.7	4.53	5.71
	D₂	a	303	98.7	62.7	39.2	25.7	16.4	8.0	4.81	5.69
	D₂	b		98.2	62.6	39.8	25.8	16.5	8.0	4.79	5.71
3	D₁	a	308	98.6	60.8	39.2	27.2	17.3	6.3	3.53	5.83
	D₁	b		99.0	60.7	39.7	27.7	17.8	6.5	3.57	5.94
	D₂	a	303	98.7	61.8	39.8	27.2	17.5	6.5	3.48	5.81
	D₂	b		98.1	60.1	39.7	27.2	17.6	6.6	3.57	5.78
4	D₁	a	278	97.6	67.7	39.6	27.8	18.6	7.7	4.17	5.69
	D₁	b		97.9	60.9	39.2	27.8	18.4	7.7	4.51	5.70
	D₂	a	277	97.5	61.9	39.5	27.8	18.6	7.7	4.49	5.93
	D₂	b		96.8	60.7	38.8	27.6	18.3	7.7	4.48	5.84
5	D₁	a	307	97.7	63.4	40.2	27.2	18.5	10.0	6.47	5.61
	D₁	b		96.8	64.5	40.1	27.4	18.7	10.0	6.55	5.63
	D₂	a	303	96.9	61.9	39.1	26.9	18.2	9.6	6.18	5.69
	D₂	b		96.6	62.1	39.2	26.9	18.2	9.6	6.10	5.70
6	D₁	a	292	93.8	57.5	39.6	26.4	17.6	8.4	5.02	5.68
	D₁	b		95.0	58.2	39.9	26.3	17.4	8.3	4.90	5.67
	D₂	a	301	95.2	58.3	40.1	26.5	17.6	8.4	4.95	5.76
	D₂	b		95.3	58.7	39.7	26.4	17.6	8.4	5.02	5.70
7	D₁	a	302	95.7	61.6	40.6	26.3	16.9	7.6	4.25	5.92
	D₁	b		95.4	60.5	40.0	26.5	17.1	7.8	4.35	5.88
	D₂	a	303	94.4	62.1	40.9	26.8	17.1	7.7	4.30	5.87
	D₂	b		95.7	57.6	39.2	25.8	16.6	7.4	4.09	5.77
8	D₁	a	298	96.5	63.2	40.1	28.1	18.5	7.6	4.03	5.77
	D₁	b		97.0	63.4	41.1	28.5	18.7	7.8	4.19	5.75
	D₂	a	298	98.1	68.2	43.8	29.7	19.4	8.0	4.32	5.91
	D₂	b		97.5	63.9	41.1	28.5	18.7	7.8	4.17	5.78
9	D₁	a	312	97.8	51.2	35.3	25.6	17.7	8.2	4.31	5.57
	D₁	b		96.8	52.7	35.6	25.8	18.1	8.5	4.55	5.65
	D₂	a	311	96.0	50.5	34.7	25.2	17.7	8.2	4.39	5.52
	D₂	b		97.6	53.6	35.9	25.6	17.7	8.2	4.36	5.63
10	D₁	a	272	92.5	56.4	39.7	26.8	18.2	8.6	4.90	5.57
	D₁	b		92.7	56.4	40.6	27.2	18.2	8.6	4.79	5.62
	D₂	a	270	93.1	60.3	42.2	27.9	18.6	8.6	4.81	5.67
	D₂	b		93.1	58.3	41.0	27.3	18.3	8.6	4.82	5.57
11	D₁	a	280	92.0	57.6	40.3	27.5	18.6	9.1	4.97	5.63
	D₁	b		94.4	58.7	40.7	27.6	18.7	9.0	5.04	5.84
	D₂	a	281	92.3	52.6	37.5	26.0	17.7	8.8	5.00	5.66
	D₂	b		95.1	58.5	40.3	27.2	18.3	8.8	4.76	5.79
12	D₁	a	300	94.4	60.8	37.3	23.1	16.4	9.8	5.64	5.80
	D₁	b		92.0	59.2	36.6	22.9	16.1	9.5	5.31	5.76
	D₂	a	301	93.3	60.9	37.0	22.9	16.3	9.8	5.63	5.76
	D₂	b		94.9	59.9	36.9	23.0	16.2	9.6	5.43	5.74
13	D₁	a	294	97.9	64.8	39.6	25.3	17.4	9.2	5.64	5.73
	D₁	b		98.1	63.2	38.9	25.2	17.3	9.1	5.58	5.53
	D₂	a	293	98.1	65.3	40.0	25.6	17.5	9.2	5.60	5.84
	D₂	b		98.1	62.2	38.8	25.3	17.4	9.3	5.68	5.63
14	D₁	a	284	97.5	64.5	40.6	27.1	18.7	9.6	5.79	6.06
	D₁	b		97.4	64.7	41.1	27.2	18.7	9.5	5.66	6.08
	D₂	a	281	98.2	62.8	39.8	26.5	18.1	9.1	5.47	5.99
	D₂	b		98.2	62.7	40.0	26.8	18.3	9.2	5.53	6.01
26	D₁	a	312	97.7	58.4	40.3	28.8	19.8	8.6	4.62	5.90
	D₁	b		98.1	60.2	40.9	29.2	20.3	9.0	5.00	5.96
	D₂	a	313	98.5	60.7	41.3	29.4	20.2	8.8	4.73	5.91
	D₂	b		97.6	59.5	40.5	29.1	19.9	8.7	4.69	5.89
27	D₁	a	298	98.8	64.3	42.2	29.7	20.5	10.0	5.78	5.86
	D₁	b		99.2	62.5	41.4	29.1	20.3	10.0	5.81	5.74
	D₂	a	295	98.5	62.9	41.3	29.1	20.1	9.8	5.55	6.04
	D₂	b		98.8	63.4	41.4	29.1	20.0	9.7	5.54	6.04
28	D₁	a	282	95.7	62.3	40.7	29.2	21.9	14.2	9.56	6.17
	D₁	b		95.4	61.8	40.2	29.0	21.8	14.1	9.45	6.21
	D₂	a	279	93.2	58.7	39.1	28.5	21.5	13.8	9.26	6.04
	D₂	b		95.2	59.5	39.2	28.7	21.7	14.1	9.55	6.01
29	D₁	a	312	98.2	61.8	37.6	25.1	17.1	8.7	5.04	5.86
	D₁	b		96.9	61.8	37.8	24.9	17.0	8.8	5.19	5.83
	D₂	a	311	96.0	58.5	36.6	24.6	16.8	8.7	5.07	5.83
	D₂	b		96.9	59.9	36.3	24.6	16.8	8.6	5.01	5.85
30	D₁	a	282	97.9	63.7	40.8	29.1	21.2	12.1	7.00	5.84
	D₁	b		98.0	64.9	41.7	29.5	21.5	12.3	7.25	5.87
	D₂	a	281	98.4	60.4	39.0	28.5	20.8	11.9	7.61	5.67
	D₂	b		97.3	60.0	38.9	28.5	20.9	11.9	7.00	5.69
31	D₁	a	322	96.0	57.3	39.6	25.8	17.7	9.1	4.95	5.86
	D₁	b		96.3	54.7	37.4	25.1	17.4	9.0	4.82	5.79
	D₂	a	321	96.0	53.6	37.1	25.1	17.3	9.0	4.89	5.69
	D₂	b		96.3	54.0	36.9	25.0	17.2	8.9	4.86	5.75
32	D₁	a	322	97.3	60.0	34.7	23.2	16.5	9.4	5.54	5.75
	D₁	b		98.6	62.8	36.2	23.8	16.8	9.6	5.74	5.90
	D₂	a	321	98.7	62.9	35.9	23.8	16.8	9.6	5.71	5.80
	D₂	b		98.9	63.5	37.2	24.3	17.2	9.9	6.02	5.82
33	D₁	a	326	99.5	69.7	38.9	24.3	16.9	9.4	5.53	6.04
	D₁	b		98.7	69.9	38.6	24.4	17.0	9.4	5.58	6.01
	D₂	a	327	98.9	70.1	38.8	24.4	17.1	9.6	5.74	5.90
	D₂	b		98.8	70.3	38.5	24.3	17.1	9.7	5.82	5.87
34	D₁	a	312	99.4	63.7	38.4	26.2	18.5	9.9	5.45	5.95
	D₁	b		98.4	66.4	39.4	26.6	18.8	10.1	5.56	6.04
	D₂	a	313	98.4	67.0	39.4	26.6	18.7	9.9	5.40	6.04
	D₂	b		99.1	63.7	38.1	26.1	18.4	9.8	5.27	5.86
35	D₁	a	330	98.5	67.9	36.9	24.0	16.9	9.4	5.49	5.89
	D₁	b		99.5	69.5	37.4	24.7	17.4	10.0	6.00	5.87
	D₂	a	330	99.2	67.0	36.7	24.3	17.2	9.8	5.89	5.96
	D₂	b		98.9	69.1	36.7	24.4	17.2	9.9	5.94	6.05
36	D₁	a	316	99.2	65.4	39.1	25.5	18.0	10.3	6.24	6.05
	D₁	b		98.3	64.2	38.4	25.4	17.9	10.3	6.17	6.01
	D₂	a	317	97.9	66.2	38.8	25.7	18.2	10.4	6.30	6.06
	D₂	b		99.4	66.0	38.8	27.4	17.9	10.3	6.18	6.01
37	D₁	a	316	98.1	58.9	40.5	26.8	19.0	10.6	5.94	6.05
	D₁	b		98.4	60.4	41.4	27.4	19.3	10.8	6.05	6.14
	D₂	a	318	98.4	59.5	40.6	26.8	18.8	10.6	5.91	6.12
	D₂	b		97.0	60.4	40.8	27.2	19.2	10.8	6.04	6.10
38	D₁	a	310	98.1	58.4	41.2	28.2	20.2	11.8	7.64	5.93
	D₁	b		95.8	57.2	39.8	27.3	19.6	11.4	7.35	5.74
	D₂	a	311	97.8	57.1	40.0	27.9	20.0	11.6	7.36	5.86
	D₂	b		96.7	56.7	39.3	27.3	19.6	11.3	7.25	5.75
39	D₁	a	305	95.2	55.0	39.7	27.9	26.4	11.8	7.28	6.02
	D₁	b		96.8	57.8	41.3	28.6	20.9	12.0	7.40	6.16
	D₂	a	306	98.0	59.7	42.8	29.2	21.3	12.4	7.76	6.02
	D₂	b		97.8	55.7	40.7	28.4	20.6	12.0	7.45	5.93

15	D_1	a	293	97.5	61.2	41.4	29.5	12.0	12.0	7.89	[2]5.63	40	D_1	a	329	96.7	66.6	37.9	25.5	18.0	9.7	5.66	5.94
	D_1	b		97.1	61.0	41.5	29.5	12.0	12.0	7.91	[2]5.63		D_1	b		96.4	63.9	27.6	25.1	17.6	9.6	5.57	6.02
	D_2	a	288	97.6	61.1	41.2	29.6	12.2	12.2	8.07	[2]5.79		D_2	a	328	98.2	63.4	37.5	25.4	17.9	9.7	5.57	6.03
	D_2	b		96.7	59.8	40.8	29.3	12.0	12.0	7.90	[2]5.72		D_2	b		97.4	63.4	37.6	27.3	17.9	9.7	5.66	5.97
16	D_1	a	291	96.7	57.1	39.2	27.8	19.4	9.5	5.49	[2]5.83	41	D_1	a	319	98.4	67.1	35.4	23.3	16.1	8.8	5.22	5.95
	D_1	b		96.9	57.1	39.0	27.5	19.2	9.3	5.44	[2]5.95		D_1	b		99.0	68.3	35.6	23.1	16.0	8.7	4.98	5.95
	D_2	a	296	98.7	60.3	40.6	28.4	19.8	9.5	5.50	[2]5.88		D_2	a	320	98.9	68.1	35.6	23.5	16.3	8.9	5.19	6.13
	D_2	b		98.1	60.7	40.5	28.3	19.7	9.5	5.50	[2]5.82		D_2	b		99.3	67.4	35.5	23.2	16.2	8.8	5.23	6.17
17	D_1	a	287	97.7	60.8	39.1	26.9	19.5	11.3	7.55	[2]5.76	42	D_1	a	325	98.5	65.7	35.7	22.8	15.6	8.4	5.01	6.02
	D_1	b		98.6	60.5	38.8	26.8	19.4	11.2	7.53	[2]5.68		D_1	b		98.9	67.9	36.0	22.9	15.6	8.5	4.95	6.05
	D_2	a	286	98.6	60.6	38.7	26.6	19.2	11.1	7.49	[2]5.71		D_2	a	325	98.2	64.4	35.6	22.6	15.4	8.3	4.93	5.99
	D_2	b		98.4	61.4	38.8	26.6	19.1	11.6	7.42	[2]5.75		D_2	b		98.1	65.5	35.4	22.6	15.3	8.2	4.93	6.03
18	D_1	a	277	94.8	58.9	42.1	29.2	20.5	10.2	5.79	[2]5.83	43	D_1	a	318	99.0	65.1	35.6	22.6	15.3	8.3	4.78	5.99
	D_1	b		96.5	59.5	42.9	29.6	20.7	10.3	5.83	[2]5.72		D_1	b		98.9	66.6	36.7	23.2	15.6	8.3	4.79	6.07
	D_2	a	271	96.0	58.6	42.1	29.2	20.6	10.3	5.83	[2]5.76		D_2	a	318	99.2	64.8	35.9	22.7	15.4	8.3	4.73	6.10
	D_2	b		96.8	56.4	41.1	28.9	20.4	10.2	5.77	[2]5.73		D_2	b		99.6	65.9	36.0	22.8	15.5	8.4	5.07	6.04
19	D_1	a	266	96.7	60.5	44.9	31.1	21.3	9.9	5.49	6.11	44	D_1	a	300	99.7	69.3	34.7	22.4	15.5	8.5	4.84	6.00
	D_1	b		96.6	58.4	44.1	30.8	21.2	9.9	5.46	6.02		D_1	b		98.8	68.1	34.5	22.3	15.4	8.4	4.79	5.96
	D_2	a	267	94.8	57.1	43.0	30.4	21.0	9.8	5.52	6.01		D_2	a	300	99.5	71.1	35.2	22.4	15.6	8.5	4.75	6.10
	D_2	b		95.1	55.7	42.2	29.9	20.7	9.7	5.45	5.90		D_2	b		99.0	67.0	34.3	22.2	15.5	8.5	4.91	5.99
20	D_1	a	310	97.9	60.2	37.8	25.9	17.6	8.5	4.97	5.96	45	D_1	a	324	99.2	63.7	37.8	25.6	18.0	10.1	6.09	5.99
	D_1	b		97.8	60.5	37.8	25.7	17.4	8.5	5.01	5.92		D_1	b		99.7	63.7	37.7	25.2	17.6	9.9	5.96	5.99
	D_2	a	307	97.8	60.5	37.8	25.7	17.3	8.3	4.88	6.05		D_2	a	324	98.3	62.1	36.6	24.7	17.3	9.6	5.74	6.07
	D_2	b		97.0	60.3	37.6	25.7	17.3	8.3	4.92	6.01		D_2	b		99.2	61.1	36.1	24.5	17.1	9.5	5.74	6.02
21	D_1	a	317	94.5	58.3	39.2	27.0	18.9	9.8	5.72	5.76	46	D_1	a	322	97.9	58.6	38.9	25.6	17.5	9.1	5.03	6.03
	D_1	b		94.6	56.5	38.7	26.6	18.5	9.4	5.42	5.84		D_1	b		97.5	59.7	39.3	25.9	17.7	9.2	5.08	6.04
	D_2	a	316	94.0	56.1	38.4	26.4	18.4	9.3	5.29	5.86		D_2	a	322	97.0	59.7	38.8	25.6	17.5	9.1	5.04	6.11
	D_2	b		96.8	57.7	38.9	27.0	18.8	9.7	5.57	5.76		D_2	b		97.1	57.1	37.4	25.1	17.3	9.0	5.03	5.94
22	D_1	a	272	96.0	61.3	39.6	27.7	19.3	9.4	5.41	6.19	47	D_1	a	292	99.5	66.4	37.4	25.1	18.0	10.7	7.02	6.03
	D_1	b		95.6	58.1	38.7	27.6	19.3	9.7	5.76	6.04		D_1	b		99.4	66.2	36.3	24.6	17.7	10.3	6.58	6.00
	D_2	a	271	95.8	57.9	38.1	27.2	18.9	9.3	5.41	6.10		D_2	a	292	99.6	67.5	37.2	24.6	17.6	10.2	6.53	6.08
	D_2	b		95.5	58.8	38.1	27.0	18.7	9.1	5.22	6.09		D_2	b		99.6	68.1	37.2	24.7	17.7	10.3	6.57	6.07
23	D_1	a	310	97.9	60.5	34.9	24.0	16.4	8.1	5.04	6.03	48	D_1	a	332	99.3	63.9	37.0	26.1	19.2	11.8	8.10	5.72
	D_1	b		98.2	61.6	34.6	23.6	15.9	7.5	4.20	6.10		D_1	b		99.7	67.7	37.8	26.1	19.2	11.8	8.10	5.83
	D_2	a	307	98.6	64.0	36.3	24.4	16.5	8.1	4.77	6.12		D_2	a	332	98.8	65.2	37.0	26.0	19.1	11.6	8.04	5.84
	D_2	b		97.6	63.1	36.0	24.4	16.5	8.0	4.74	6.16		D_2	b		99.8	64.8	36.8	25.8	18.8	11.3	7.73	5.83
24	D_1	a	322	98.5	65.1	39.1	27.7	19.3	9.3	5.23	5.89	49	D_1	a	344	98.9	63.8	40.1	28.2	20.2	11.0	6.48	5.84
	D_1	b		98.2	65.2	38.8	27.4	18.9	9.0	5.62	5.80		D_1	b		98.7	63.8	39.6	28.0	19.9	10.8	6.25	5.81
	D_2	a	321	98.4	68.5	40.5	28.0	19.3	9.2	5.69	5.95		D_2	a	345	98.8	67.3	43.9	29.7	20.8	11.3	6.63	6.04
	D_2	b		99.0	67.1	39.5	27.7	19.1	9.1	5.68	5.95		D_2	b		97.6	65.7	42.8	29.1	20.5	11.2	6.56	5.91
25	D_1	a	320	98.1	62.4	41.7	29.2	19.9	8.9	4.87	5.78	50	D_1	a	334	99.4	69.3	40.1	27.1	19.1	10.0	5.60	5.79
	D_1	b		98.1	60.3	40.4	28.7	19.4	8.5	4.70	5.81		D_1	b		99.3	71.3	41.7	27.9	19.7	10.3	5.87	5.71
	D_2	a	315	98.2	60.0	39.7	28.5	19.4	8.7	4.85	5.83		D_2	a	335	99.1	68.0	39.4	26.9	19.0	9.9	5.49	5.74
	D_2	b		97.9	57.0	39.0	28.2	19.2	8.6	4.76	5.79		D_2	b		99.2	72.8	42.5	28.0	19.6	10.3	5.80	5.80

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[1] For all samples, 100 percent of the aggregate passed the ½-inch sieve.

[2] Change in plant target value (JMF) for asphalt content ordered by project engineer on days these samples were taken. Therefore, a correction, 0.20, was added to these prior to statistical treatment.

Table 4.—Analysis of variance-extraction test results

Test property	Number of tests	Variance—				Standard deviation
		Process	Sampling	Testing	Total	
Mix temperature ° F	100	341.4	[2] 1.9	--------	343.3	18.0
Asphalt content [1] percent	200	0.00864 [3] 0.093	0.00234 [3] 0.048	0.00339 [3] 0.058	.01438	0.120
Gradation, percent passing sieve:						
⅜-inch percent	200	2.507 [3] 1.567	0.0185 [3] 0.136	0.4277 [3] 0.654	2.9527	1.718
No. 4 do	200	14.863 [3] 3.855	0.6245 [3] 0.790	1.9428 [3] 1.394	17.4305	4.175
No. 10 do	200	3.7528 [3] 1.937	0.3544 [3] 0.595	0.4115 [3] 0.641	4.5186	2.126
No. 20 do	200	4.0376 [3] 2.008	0.0566 [3] 0.238	0.0877 [3] 0.296	4.1819	2.045
No. 40 do	200	2.7892 [3] 1.670	0.0216 [3] 0.147	0.0411 [3] 0.203	2.8519	1.689
No. 80 do	200	2.1103 [3] 1.453	0.0101 [3] 0.100	0.0199 [3] 0.141	2.1402	1.463
No. 200 do	200	1.3193 [3] 1.149	0.0026 [3] 0.051	0.0172 [3] 0.131	1.3390	1.157

[1] Percent of mix.
[2] Sampling and testing.
[3] Square root for preceding result of analysis of variance.

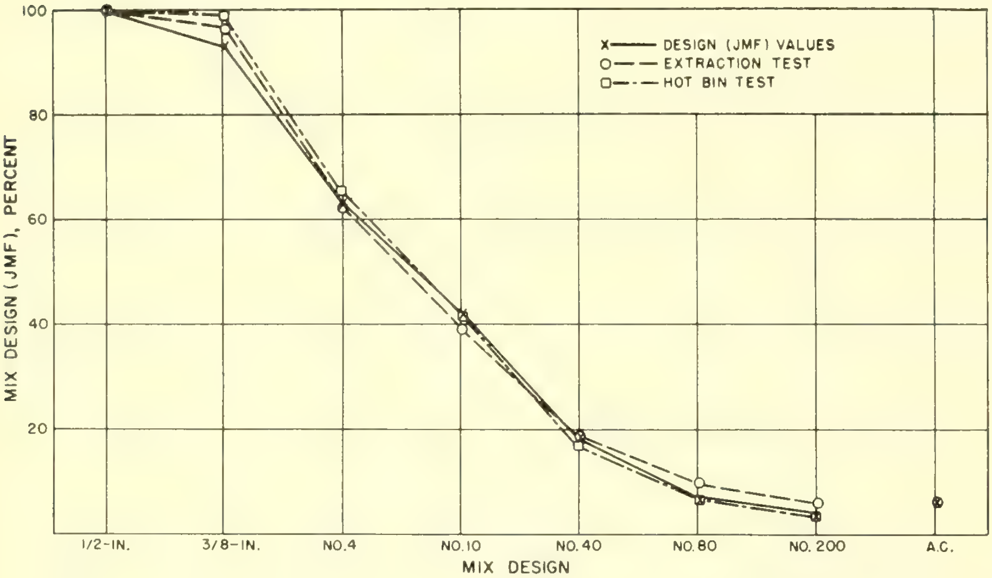

Figure 3.—Averages of test results and their relation to mix design (JMF) values.

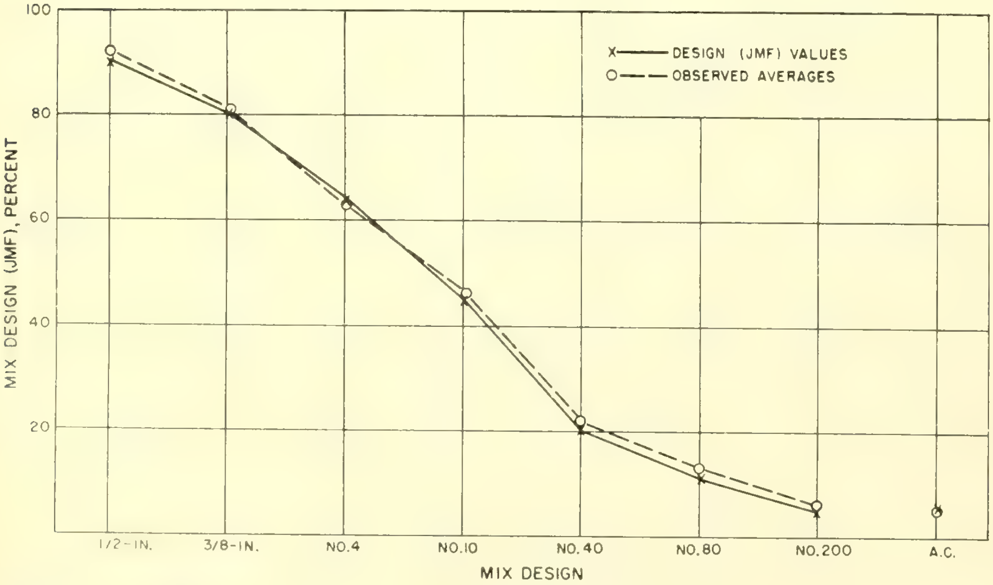

Figure 4.—Averages of extraction test results from AASHO Road Test and their relation to design (JMF) values.

by the plant inspector during normal plant control work on the sampling days covered by the study reported here. Although not shown in the table, all of these test results were within the ranges permitted by the job mix formula. A complete tabulation of the individual extraction test results is given in table 3.

The results of the analysis of variance are given in table 4. Total variance and its calculated components attributed to the plant process, sampling procedure, and test method are shown.

The percentages of all extraction test results for each test property lying within given ranges of their respective averages compared to theoretical percentages for the Normal Curve (*5*) are given in table 5. The close agreement of the observed percentages with the theoretical percentages indicated that the data were essentially normal and, therefore, amenable to statistical treatment.

Plots of cumulative frequencies on normal probability paper also indicated that the data were essentially normal in distribution, except for the percentage of aggregate passing the No. 200 sieve. The data for this sieve size showed some deviation from a normal distribution—starting at the 87th percentile. However, this moderate deviation was not considered sufficient to invalidate the statistical treatment.

Averages

Data for comparing the averages of the extraction and hot bin test results with the mix design for each element of the job mix formula are shown by curves in figure 3. The extraction test averages conformed closely to the design for each item. In the extraction test results, the averages for aggregate passing the ½-inch, No. 4, and No. 40 sieves, and the asphalt content were within 1 percentage point of their respective design values. The largest departure from design, was the 4.5 percentage obtained for aggregate passing the ⅜-inch sieve. Also shown in figure 3, are the averages obtained in the hot bin gradation control tests. These averages, although based on only 10 test results, had generally good agreement with the mix design. Compared with the extraction test results, the hot bin averages were somewhat larger for the larger sieve sizes (No. 20 and larger) and lower for the smaller sieve sizes (No. 40 and less). Such differences were not unexpected because of the differences in handling and the test methods used for the aggregates. The aggregates obtained in the extraction tests had been subjected to pugmill mixing and centrifuging, whereas the hot bin aggregates had not been subjected to either of these influences. Furthermore, the extracted aggregates were sieved wet and the hot bin aggregates were sieved dry in accordance with the normal control practice used in the construction project.

The mix design and the corresponding averages obtained by extraction tests for surface course mixtures used in the AASHO Road Test (*6*) are shown in figure 4. A comparison of figures 3 and 4 shows that, for able 5.—Observed percentages and theoretical percentages of test results within given ranges

Range, $\bar{X} \pm t \times$ S.D.	Theoretical percentage within range	Observed percentages within range for data for—							
		Aggregate passing sieve sizes—							Asphalt content
		⅜-in	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200	
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
$\bar{X} \pm 0.675 \times$ S.D.	50.0	52.5	51.0	49.5	49.5	52.5	55.0	59.5	45.5
$\bar{X} \pm 1.0 \times$ S.D.	68.3	74.5	68.5	67.0	64.5	65.5	71.5	75.5	66.5
$\bar{X} \pm 1.5 \times$ S.D.	86.6	91.0	89.5	85.5	84.0	75.5	86.5	85.0	85.5
$\bar{X} \pm 2.0 \times$ S.D.	95.5	94.5	96.5	95.0	98.5	96.0	94.5	96.0	96.5
$\bar{X} \pm 2.5 \times$ S.D.	98.7	96.5	98.5	99.5	100.0	100.0	98.0	98.0	99.0
$\bar{X} \pm 3.0 \times$ S.D.	99.7	99.0	100.0	100.0	100.0	100.0	98.5	98.0	100.0

verages obtained in extraction tests, the verall conformance to design for the mix prouction studied was comparable to that btained in the AASHO test. Because of resent tendencies to suspect the accuracy of n extraction test for the determination of sphalt content of mixtures, the close agreeient, shown in table 1 and figure 3, between ne determined average 5.955 percent and the esign or target percentage of 6.0 percent is of nterest. This close agreement is believed to ave been the result of close adherence to the xtraction test method (*2*) combined with ccurate plant scales and proportioning, good ampling technique, and favorable asphalt-ggregate interfacial conditions.

ariation in individual results

The standard deviations shown in tables 1 nd 2 indicate the variations in individual esults obtained for each property by mix xtraction tests and by hot bin gradation ests, respectively. A comparison of these ariations by the statistical *F* test at the 5-ercent level showed no significant difference n variability between the two sets of results xcept for the ⅜-inch and the No. 4 sieves. ariation, expressed in percentages of indiidual results within and outside design ange, is shown in the last three columns of able 1. These percentages might be conidered another indication of the quality of onformance to the design, for each item hown. For three of the eight items specified or mix composition—percentage of aggregate assing the No. 10, No. 80, and No. 200 ieves—10 percent or more of the results exeeded the design limits. These departures rom 100 percent conformance are explained n the footnotes to the table. From the averges and the standard deviations for these tems, the deviations seem to have been the esult primarily of the use of aggregate that ested too close to either the upper or lower imits of the ranges set in the job mix formula.

The large percentage, 30 percent, of mix emperatures outside the permissible limits vas caused, at least in part, by the intentional hanges in plant operating temperatures to compensate for those day-to-day fluctuations in atmospheric temperatures that seemed to affect compaction.

Table 6.—Comparison of variations—standard deviation

Source of data	Study for this article	AASHO (Ref. 6)	WASHO (Ref. 7)
Number of tests	200	96	72
Asphalt content percent	0.120	0.18	0.40
Gradation, aggregate passing sieve:			
⅜-inch percent	1.72	3.17	
No. 4 do	4.18	4.06	4.36
No. 10 do	2.13	2.99	2.81
No. 20 do	2.05	1.68	..
No. 40 do	1.69	2.06	1.69
No. 80 do	1.46	1.07	-
No. 200 do	1.16	1.16	0.69

Many previous investigators have reported variation in asphalt concrete composition. The variation results from two such investigations—the AASHO Road Test (*6*) and the earlier WASHO Road Test (*7*)—and the results from the study reported here are indicated by the standard deviations shown in table 6. Evaluations of these results by Bartlett's Test (*8*) showed that, except for aggregate passing the No. 4 and No. 40 sieves, the differences in variability between the data sources were statistically significant. Thus the observed variations in asphalt content and in aggregate passing the ⅜-inch, No. 4, No. 10, and No. 40 sieves were equal to or less than those that were obtained in the AASHO and WASHO tests.

Causes of variation

The results of the analyses of variance are summarized in table 4, giving overall or total variance and the components of variance for each item. Although variance is the basic statistical measure of variability (*8*), its computation involves an arithmetic squares of the original units. It is therefore customary to take the square root of a variance in order to obtain results in units on the same scale as the original. The results thus obtained are standard deviations. These are shown in the table just below the entry for each variance component. The square root of total variance is given separately under standard deviation. These results show that inherent process variation was greater than variation attributed to sampling or testing. As variances are additive quantities, it is apparent that for each item process variance was greater than variance from sampling procedure and test method combined.

The relatively large effect of process variability on overall variation indicates that future efforts to improve the uniformity of the plant's product should logically begin with the plant process. Although it is recognized that process variability as determined in the study reported here included several factors, the problem of identifying the significant causative ones may not be as formidable as may seem at first. For instance, helpful clues to improvement might be furnished by the plant's personnel who probably know from experience which phases—aggregate source, stockpile uniformity, cold bin loading, hot bin screens, etc.—of the plant process are likely to be suspect. However, lacking such prior information, a modest test program covering a few phases of the plant's process might be all that is necessary to locate the real cause of undesirable product variation so that specific corrective action might be suggested.

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(*4*) *A Plan for Expediting the Use of Statistical Concepts in Highway Acceptance Specifications*, (M–W Technical Report No. 201) by Miller-Warden Associates, Aug. 1963.

(*5*) *ASTM Manual on Quality Control of Materials*, by ASTM Committee E–11 on Quality Control of Materials, Jan. 1951, p. 25.

(*6*) *The AASHO Road Test—Report 2, Materials and Construction*, HRB Special Report 61B, 1962.

(*7*) *The WASHO Road Test—Part 1: Design, Construction, and Testing Procedures*, HRB Special Report 18, 1954.

(*8*) *Statistical Methods for Chemists*, by W. J. Youden, 1951.

Method of Computing Analysis of Variance

To compute the variances proceed as follows:

Step 1.—Construct form similar to the sample worksheet, figure 5, with spaces in the left column equal to the number of units to be sampled. Assign an identification number to each unit sampled.

Step 2.—Identify the duplicate samples taken from each unit as D_1 and D_2. Identify the two portions obtained by splitting each duplicate sample as a and b. Each unit is now represented by 4 portions D_1a; D_1b; D_2a; D_2b.

Step 3.—List the values obtained from measurement or testing in columns (1) and (2) in the spaces provided for the 4 portions representing the unit identified by the number in the left column.

Step 4.—In column (3) list the difference between columns (1) and (2) for both the D_1 and D_2 duplicate samples.

Step 5.—In column (4) list the sums of columns (1) and (2) for both the D_1 and D_2 duplicate samples.

Step 6.—In column (5) list the differences between the D_1 and D_2 duplicate samples of each unit that appear in column (4).

Step 7.—In column (6) list the sums of the D_1 and D_2 duplicate samples of each unit that appear in column (4).

Step 8.—Total all the numbers that appear in columns (1) and (2). Then total the numbers in columns (4) and (6). If no error has been made, all totals will be the same. Enter totals in blocks A for each column in row (7) Totals.

Step 9.—Square each number in column (3) and total these squares. Enter this total in block B in column (3) in row (8) Sums of Squares. Repeat for columns (5) and (6) and enter sums in blocks C and D.

Step 10.—Record *n*, the number of sampled units appearing in the left column in row (9).

Step 11.—Compute $\frac{A^2}{4n} = CF$ (correction factor) and enter in row (10).

Step 12.—Compute unit sum of squares $\frac{D}{4} - CF = V_1$. Enter result in row (11).

Step 13.—Compute duplicate sample sum of squares $\frac{C}{4} = V_2$ and enter result in row (12).

Sample number	Duplicate number	Column (1) a	Column (2) b	Column (3) (1) - (2)	Column (4) (1) + (2)	Column (5) $(4D_1)-(4D_2)$ Duplicate difference	Column (6 $(4D_1)+(4D_2$ Duplicate sum
1	D_1						
	D_2						
2	D_1						
	D_2						
3	D_1						
	D_2						
50	D_1						
	D_2						
(7) Totals			A		A		A
(8) Sums of squares				B		C	D

		Analysis of Variance				
		Source	d.f.	Sum of squares	Mean squares	Variance componen
(9)	n					
(10)	$CF = \frac{A^2}{4n}$	Samples	n-1=	V_1=	MS_1=	σ_a^2=
(11)	$V_1 = \frac{D}{4} - CF$	Duplicates	n=	V_2=	MS_2=	σ_s^2=
(12)	$V_2 = \frac{C}{4}$	Portions	2n=	V_3=	MS_3=	σ_t^2=
(13)	$V_3 = \frac{B}{2}$					σ' =

Figure 5.—Suggested worksheet for computing analysis of variance.

Step 14.—Compute sum of squares for test portions $\frac{B}{2} = V_3$ and enter result in row (13).

Step 15.—Fill in analysis of variance table using value of *n* from row (9); V_1 from row (11); V_2 from row (12); and V_3 from row (13).

Step 16.—Compute mean squares (MS) by dividing sum of squares (SS) by degrees of freedom (d.f.).

Step 17.—Compute variance components:

$\sigma_t^2 = MS_3$

$\sigma_s^2 = \frac{MS_2 - MS_3}{2}$

$\sigma_a^2 = \frac{MS_1 - MS_2}{4}$

Record these values in lower right corner o worksheet. If any value is negative, recor as zero.

Step 18.—Compute overall sigma (σ'):

$\sigma' = \sqrt{\sigma_a^2 + \sigma_s^2 + \sigma_t^2}$

Traffic Flow Responses to Unannounced Increases in Progression Speeds of Signal Systems

(Continued from p. 4.)

o that of the 10th, 20th, and 35th vehicles n the platoon for each speed of progression. The maximum difference between any two of these vehicles for each speed of progression was 1 m.p.h. As this difference is small, the unning speed of the lead vehicle was conidered representative of the running speed of all vehicles in the platoon.

Effect of lapse of time

One method of evaluating drivers adaptation to a change in the speed of progression s to analyze the daily increases in running peed through the test site after the change has been made. If adaptation is occurring, a trend should be identifiable by a regression nalysis. First, second, and third degree urves were fitted to the data for two traffic low conditions: 2,000 and 3,300 vehicles per hour. It was determined that a straight line best fit both sets of data but additional tatistical tests indicated that the slope of he lines was not statistically significant from zero. Figure 9 is a scatter diagram for 2,000 vehicles per hour. At a flow of 2,000 vehicles per hour evidence is strong that adaptation was being made despite the lack of statistical significance. The evidence justifies additional study with this technique as it may be possible that better experimental control of the variables plus an increase in sample size might permit the establishment of a statistically significant trendline. For the larger raffic flow of 3,300 vehicles per hour no adaptation to the increase in the speed of progression was evident. This seems to conirm that the volume of traffic provides the imiting factor in the speed selected by the driver.

Headways

No significant difference in time headways was detected when the speed of progression was increased. This probably is the result of he large time variation between platoons and possibly the inability of the observer technique to detect small changes in average time headways. Headway data from the traffic analyzer will be analyzed later and it is assumed that this data will show a difference n the headway distribution for the different speeds of progression.

Table 3.—Average running speed of vehicle by position in platoon [1]

Speed of progression, m.p.h.	Vehicle position in platoon			
	1st	10th	20th	35th
27	27.0	27.3	27.3	26.4
33	29.5	29.2	28.6	28.5
40	29.1	29.5	30.0	29.5

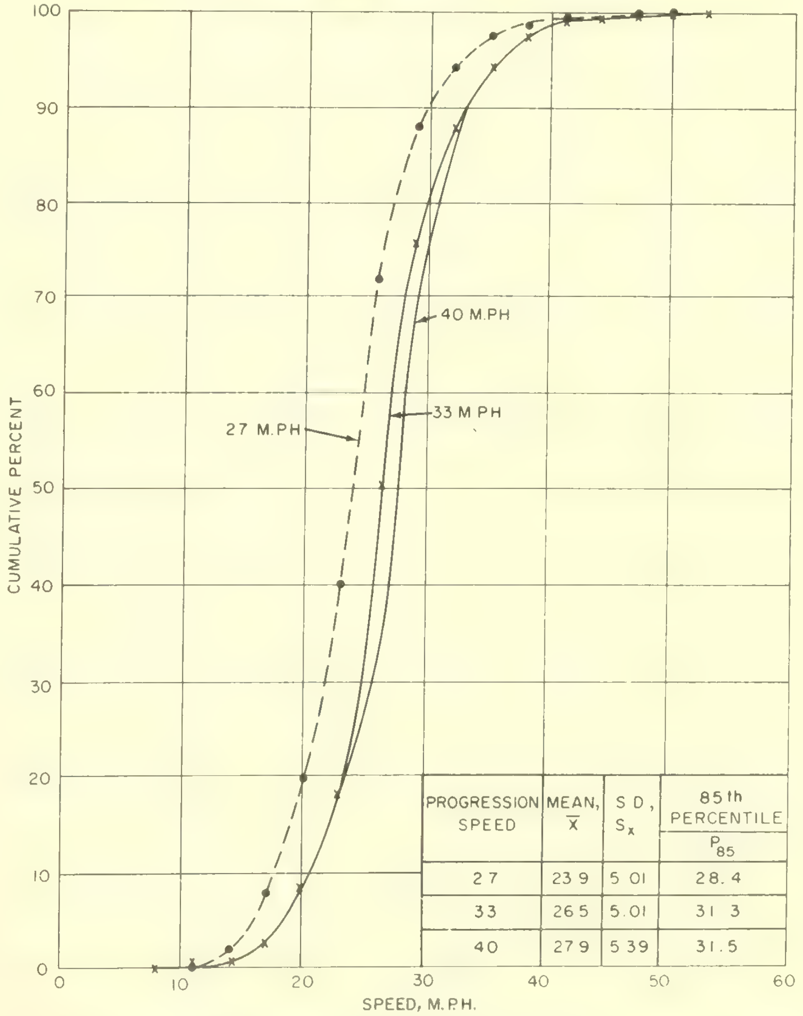
[1] This is the average of the running speeds between observers 1 and 2, 2 and 3, and 3 and 4.

Vehicle Stops

If the traffic signals are set at a certain speed of progression, and to the extent that drivers do not adapt to this speed, an increased number of vehicles will be stopped on the artery within the effective limits of the signal progression. An investigation of this extra stopping was not an objective of the study reported here, but the data do provide some insight into the relative magnitude of the number of stops attributable to the three speeds of progression investigated.

The number of stops made in the test site was estimated by using the volume counts made by the observers to compute the loss in platoon volume through the test site. This loss was adjusted for: (1) the number of vehicles overtaken by the platoon, (2) the expected number of turning vehicles, and (3) the number of vehicle stops that were attributable to emergency vehicles, police cars and tow trucks, and passenger pickups. The net adjusted loss was assumed to represent vehicles stopped for lack of driver adaptation to the effective speed of progression. Although additional factors might have accounted for minor changes in platoon size, it is assumed that any major change reflects the lack of adaptation to the change in the speed of progression. In other words, drivers who do not increase their speed relative to an increased progression speed in the signal system will be delayed by stops more frequently than those who do.

The percentage of vehicles stopped at each speed of progression is shown in figure 10 as a function of the hourly volume. Up to a flow



PROGRESSION SPEED	MEAN, $\bar{X}$	S D, S_x	85th PERCENTILE P_{85}
27	23.9	5.01	28.4
33	26.5	5.01	31.3
40	27.9	5.39	31.5

Figure 7.—Distribution of spot speeds for all lanes for three speeds of progression.

of 2,000 vehicles per hour, little difference occurred in the percentage of stops at any of the three progression speeds. At a flow of 2,500 vehicles per hour, the stopping percentage was negligible for the 27 m.p.h. speed of progression, but the percentages increased to 16 and 28 percent respectively for the 33 m.p.h. and 40 m.p.h. speeds of progression. When the flow of traffic was more than 2,500 vehicles per hour, the percentage of stopping increased for all speeds of progression. This large percentage of vehicles stopped is not alarming as the stopping was hardly noticeable on the street. On the average, the effect of the 40 m.p.h. speed of progression was to stop half of the vehicles, once each, within a test section of 3,000 feet. Stops were distributed uniformly at the six signalized intersections within the test site and had little adverse effect on the individual driver.

Economic Considerations

Data collected were insufficient to permit a comprehensive analysis of the economic factors but they do provide some insight into the relative costs and benefits for the three progressions of speed studied. Data were obtained on the running speed and the number of vehicles stopped within the test site. Except for stops, no data were obtained for speed changes, which were not considered in the analysis. As the test site essentially was free of vertical and horizontal curvature, it was assumed that the test site was level and tangent. And, as more than 98 percent of the traffic was passenger vehicles, it was assumed that all the traffic was passenger cars.

The 2.2-hour period selected for analysis was the actual length of time (7:18 to 9:30 a.m.) that data were collected for each speed of progression. In this time period, 6,070 vehicles were observed. Results of the analysis are presented in table 4. As the speed of progression was increased from 27 m.p.h. to 33 m.p.h. and then to 40 m.p.h., the total operating costs also increased.

As expected, the running time of the vehicles decreased with an increase in the speed of progression. Less expected, however, was the effect of stopped time on the total traveltime. When the speed of progression was increased from 27 to 33 m.p.h. a net decrease in traveltime of approximately 2.3 hours was obtained but a further increase in the speed of progression to 40 m.p.h. actually caused a net increase in traveltime of approximately 2.3 hours.

In the analysis no dollar value was assigned to time savings as they were considered a benefit rather than a cost item. Increase in the speed of progression from 27 to 33 m.p.h. was a benefit in time saved of approximately 2.3 hours for the analysis period, but this benefit was realized at the expense of an increased operating cost of approximately $12. Final decision concerning the optimum speed of progression could be made only after considering all engineering, nonmarketable economic items, and administrative considerations.

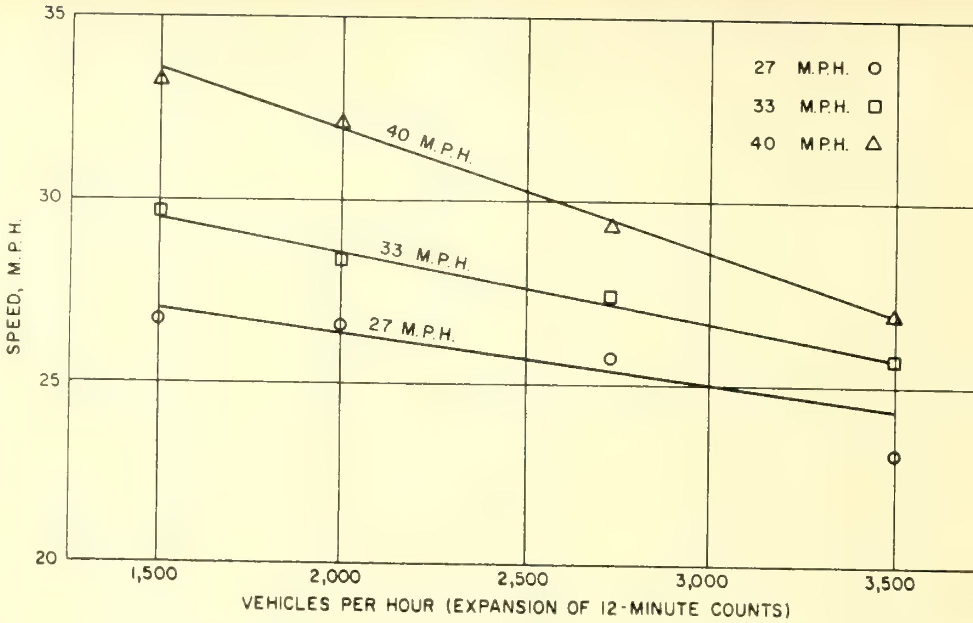


Figure 8.—Spot speed related to traffic volume for three speeds of progression.

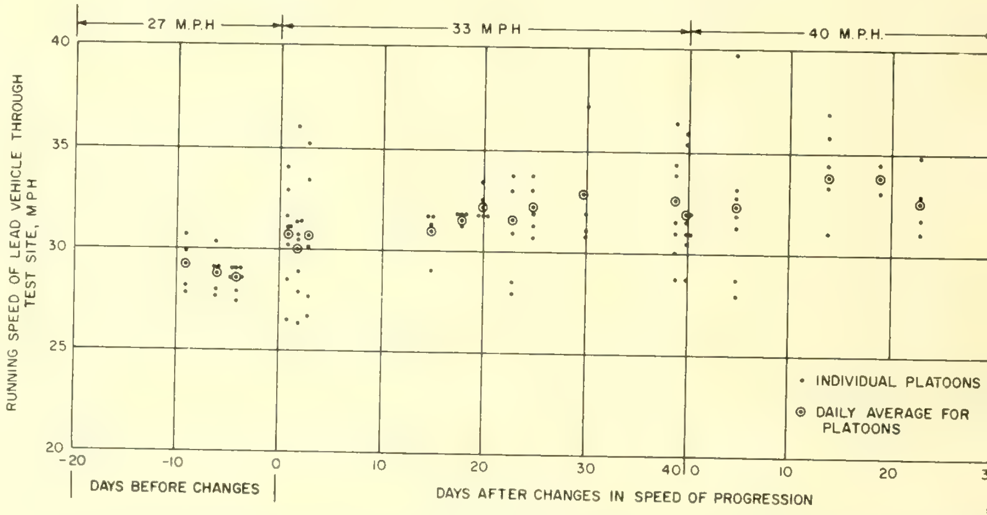


Figure 9.—Scatter for speed of lead vehicle related to days after change in progressio speed for time period 8:45 to 9:30 a.m. when traffic volume was approximately 2,00 vehicles per hour, expansion of 12-minute counts.

Evaluation of Study Techniques

Prior to conducting the study, the authors were unable to determine whether speed or headway data would be more indicative of driver adaptation to a change in the speed of progression. Further, it was not known which of the techniques available would be best suited for a study of this type. Therefore, three techniques were utilized for collecting data: the Public Roads traffic analyzer, observers stationed along the roadway, and a modified version of the floating car speed and delay study. Jointly, these techniques provided a variety of speed and headway information that could be analyzed to determine which technique and parameter were the best indicators of driver adaptation.

Traffic analyzer

The traffic analyzer, as used in the study, provided spot speed, volume, and headway information by lanes for each test condition. With this technique, data were collected at

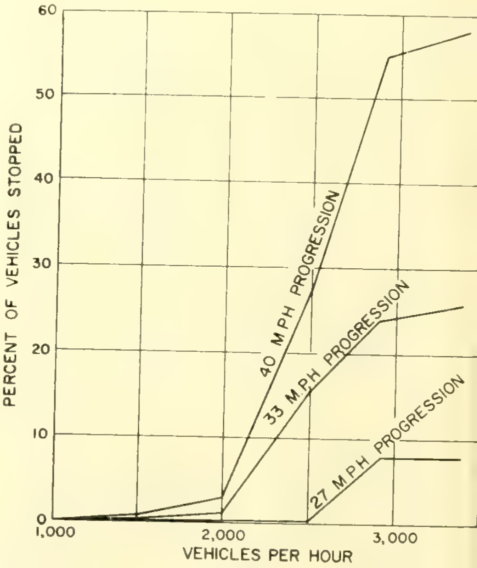
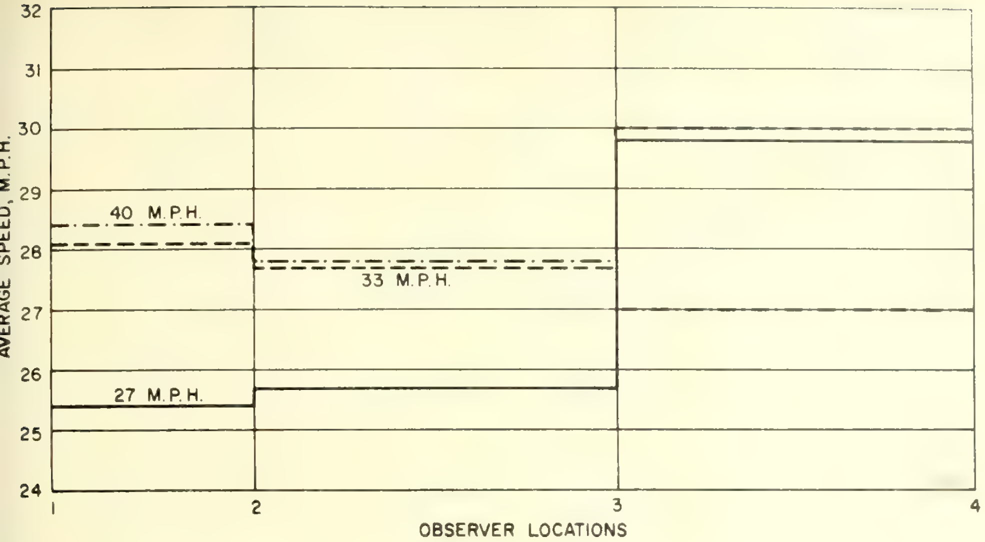


Figure 10.—Percent of vehicles stopped at each speed of progression as a function of volume.



igure 11.—Average running speed of lead vehicle between observer locations for three speeds of progression.

Table 4.—Analysis for economy [1]

Item of cost	Speed of progression—		
	27	33	40
	M.p.h.	*M.p.h.*	*M.p.h.*
Running cost at uniform speed on level tangents	$111.09	$109.49	$109.59
Excess cost of stops [2]	2.30	10.16	21.94
Cost of idling engine of stationary vehicle	.72	3.20	6.54
TOTAL COST	$114.11	$126.19	$138.07
Running time through test site ....hours	114.944	107.389	105.222
Excess consumed per speed change ....hours	.630	2.789	5.698
Time consumed while stopped .hours	.917	4.056	7.892
TOTAL TIME ...do	116.491	114.234	118.812

[1] The figures were provided from studies reported in Motor Vehicle Running Costs for Highway Economy Studies, by Robley Winfrey, Nov. 1963.

[2] Excess cost above continuing at initial speed.

only one point along the street and may not be representative of the total test site. Evidence that this may be so is shown in figure 11. The analyzer was located south of Lamont Street, which was in the test site between observers one and two. For the 27 and 33 m.p.h. speeds of progression, different spot speeds would have been observed had the analyzer been located between observers three and four. This deficiency can be overcome in the future by placing the analyzer at several locations within the test site and using an average of the spot speeds as representative of speeds on the entire test site.

Speed and delay vehicle

The modified version of the floating car speed and delay technique permitted running speed information to be collected over the distance of the test site. This had the effect of averaging the differences in running speed over short subsections of the site, which had different traffic-related characteristics. No headway data were obtained with this technique.

Observers

The running speed of the vehicle through the entire test site was obtained by calculating the elapsed time between observers at the ends of the test site. However, a marked car in the stream of traffic was required to permit identification of the lead vehicle. The headway information obtained did not show any differences in average headway for the three speeds of progression tested. This lack of differences was attributed to the large variability in average platoon headways and inaccuracies in recording the time headways by this technique.

Vehicle stoppage

There was some indication that adaptation patterns can be obtained by analyzing the stopping characteristics for vehicles at one or more intersections along the street. Lack of adaptation would be indicated by an increase in the number of vehicles stopping immediately after the change in progression speed. Driver adaptation, if present, would be indicated by a gradual decrease with time in the number of vehicles stopping.

Preferred technique

The authors determined that the floating car technique and analyzing the stopping characteristics of cars seem to be the best methods of measuring driver adaptation to changes in the speed of progression of a coordinated signal system. Each of these techniques can be used by one person and the expense will be relatively small. However, the floating car method was preferred because detection of the drivers adaptation seems to be more certain. Others planning to use this method should be certain that the sample size studied is large enough to permit detection of significant differences at the level of confidence selected, if, in fact, differences do exist. Acceptable results may also be obtained by recording traffic analyzer data at more than one location within the test site.

Limitations of the Study

The study reported here was somewhat limited by several factors. Although the speed limit was officially changed for the successive speeds of progression, it was not posted. Many drivers undoubtedly believed the speed limit to be that prevailing throughout much of the city—25 m.p.h. This belief probably affected the willingness of some drivers to increase their speeds in relation to the changes in progression speed. No knowledge by the drivers of the changes made in the speed of progression also was a limiting factor. The physical characteristics of the test site, such as the heavily signalized section from Kenyon Street to Harvard Street, accompanied by relatively frequent turning movements of the traffic, could have placed considerable restriction on speed increases, particularly at the 40 m.p.h. speed of progression.

Estimated Travel by Motor Vehicles in 1964

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

By THEODORE S. DICKERSON, JR., Highway Engineer, Current Planning Division

Motor-vehicle travel in the United States in 1964 totaled 841.9 billion vehicle-miles, an increase of 5.1 percent over the travel in 1963. The travel data were compiled by the Bureau of Public Roads from information supplied by the State highway departments and toll authorities. Total travel for 1965, based on information for the first 10 months of the year, is estimated at 880 billion vehicle-miles, a 4.5-percent increase over 1964.

The proportions of travel by road system and by vehicle type changed very little from 1963 to 1964. Of the 1964 travel, 37.6 percent was on main rural roads comprising 14 percent of the Nation's total of 3.6 million miles of roads and streets. Some 48.0 percent of the travel was on urban streets, which also comprise 14 percent of the total mileage. Travel on local rural roads was only 14.4 percent, although these roads are 72 percent of the total mileage.

Passenger cars represented 84 percent of the vehicles registered and accounted for 82 percent of the travel in 1964; trucks and truck combinations accounted for 16 percent of the vehicles and 18 percent of the travel; similar figures for buses were less than 1 percent.

Average vehicle performance in 1964 differed very little from that reported for 1963. The average motor vehicle traveled 9,644 miles in 1964, almost half of it in cities, and consumed 778 gallons of fuel at a rate of 12.40 miles per gallon, as compared with 1963 figures of 9,590 miles, 772 gallons, and 12.42 miles per gallon, respectively. The average passenger car traveled 9,417 miles and consumed 657 gallons of fuel at a rate of 14.34 miles per gallon, in 1964; comparable 1963 figures were 9,378 miles, 652 gallons, and 14.37 miles per gallon.

The travel and related information for 1964 are shown on table 1 by road system and vehicle type. Such data have been reported in PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH for a number of years, the latest for 1963 appeared in vol. 33, No. 7, April 1965, pp. 148–150.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1964 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road	Local rural road	Total rural	Urban	Total			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gallon*
Passenger cars [2]	246, 850	94, 853	341, 703	345, 432	687, 135	72, 970	9, 417	47, 924	657	14. 34
Buses:										
Commercial	908	181	1, 089	1, 803	2, 892	82. 3	35, 140	619	7, 521	4. 67
School and nonrevenue	674	743	1, 417	307	1, 724	223. 1	7, 727	241	1, 080	7. 15
ALL BUSES	1, 582	924	2, 506	2, 110	4, 616	305. 4	15, 115	860	2, 816	5. 37
All passenger vehicles	248, 432	95, 777	344, 209	347, 542	691, 751	73, 276	9, 440	48, 784	666	14. 18
Trucks and combinations	68, 180	25, 416	93, 596	56, 562	150, 158	14, 019	10, 711	19, 117	1, 364	7. 85
All motor vehicles	316, 612	121, 193	437, 805	404, 104	841, 909	87, 295	9, 644	67, 901	778	12. 40

[1] For the 50 States and District of Columbia, includes minor revisions to previously released 1964 figures.
[2] Includes taxicabs; also 985,445 motorcycles, which are estimated to account for 0.4 percent of the total travel.

NEW PUBLICATIONS

Two new publications have been issued recently by the Office of Planning, Bureau of Public Roads, U.S. Department of Commerce. Both are for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402. The price for each is $1.00. A brief discussion of each of the new publications is presented in the following paragraphs.

Traffic Assignment and Distribution for Small Urban Areas

The *Traffic Assignment and Distribution for Small Urban Areas* manual explains and illustrates the theory and use of a system of analytical procedures and computer programs for assigning and distributing trips to transportation systems with the IBM 1620 (60 K) electronic computer. The procedures described are designed to process a basic set of trip cards. They allow the computation of surveyed trip length frequency, distribution of trips between zones by the gravity model formula or Fratar method, and assignment of trip interchanges to an existing transportation network. The procedures also accommodate estimates of future trip production and trip attraction and provide for assigning the trip distribution to a proposed transportation network.

The battery of computer programs was developed by several State highway departments and the Bureau of Public Roads and is generally applicable to urban areas having populations up to 150,000 persons.

Calibrating and Testing a Gravity Model for Any Size Urban Area

The volume on *Calibrating and Testing a Gravity Model for Any Size Urban Area* is a revision of the manual of the same title published in July 1963. In this new edition, the model calibration discussions have been revised to bring them in line with current practice. Also, the program descriptions have been completely rewritten to correspond to the most current computer programs. The manual documents in detail the process of trip distribution utilizing the gravity model. The computer programs that are described in the text are, with the exception of two peripheral IBM 1401/1410 programs, designed for use on the high speed binary IBM 7090/7094 computer.

A companion publication, *The Traffic Assignment Manual,* was published by the Bureau of Public Roads in June 1964. Together these manuals document two of the basic steps necessary for transportation analysis and forecasting.

U.S. GOVERNMENT PRINTING OFFICE 1966

PUBLICATIONS
# of the Bureau of Public Roads

A list of the more important articles in Public Roads *and title sheets for volumes 24–33 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

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REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:
  Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.
  Supplementary Report, House Document No. 124 (1965). $1.00.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42 (1965). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61. $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Calibrating and Testing a Gravity Model for Any Size Urban Area (1965). $1.00.

Capacity Charts for the Hydraulic Design of Highway Culverts (Hydraulic Engineering Circular, No. 10) (1965). 65 cents.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Design of Roadside Drainage Channels (1965). 40 cents.

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Highway Bond Financing . . . An Analysis, 1950–62. 35 cents.

Highway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (1964). $1.00.

PUBLICATIONS—Continued

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (May 1965). 75 cents.

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Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). Out of print. Appendix, 70 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
  Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

Opportunities for Young Engineers in the Bureau of Public Roads (1965). 30 cents.

Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

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Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways (1958): a reference guide outline. 75 cents.

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Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents 100 for $15.00. 11 x 17, 10 cents 100 for $5.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

Traffic Assignment and Distribution for Small Urban Areas (1965). $1.00.

Traffic Assignment Manual (1964). $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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VOL. 34, NO. 2 JUNE 1966

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
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OF PUBLIC ROADS,
U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

Carson Pass Highway, California State Route 88, a Federal-Aid Primary Highway, between French Meadows Road at East of Silver Lake in Amador County, near Sacramento.

IN THIS ISSUE:

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 34, No. 2 *June 1966*

Published Bimonthly

Muriel P. Worth, *Editor*

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 16, 1966.

Contents of this publication may be reprinted. Mention of source is requested.

U.S. DEPARTMENT OF COMMERCE
JOHN T. CONNOR, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

Properties of Highway Asphalts—Part III, Influence of Chemical Composition

By [1] WOODROW J. HALSTEAD, Chief, Materials Division, Office of Research and Development, BUREAU OF PUBLIC ROADS; and FRITZ S. ROSTLER and RICHARD M. WHITE, Materials Research and Development [2]

Chemical analysis and laboratory abrasion tests were made on the 60–70 and 120–150 penetration grade asphalts, which are a part of the 323 samples collected in 1954 and 1955 by the Bureau of Public Roads for study. Results of other studies on these same asphalts have been reported previously. Also studied were a group of viscosity-graded asphalts now being used for research by other agencies. The Rostler-Sternberg precipitation method was used for the chemical analysis, and the pellet test developed by Rostler and White was used for the abrasion tests.

Of particular interest was the relation between the compositional parameter $(N+A_1)/(P+A_2)$ to durability as measured by the pellet test. The influence of consistency or viscosity of the asphalts on the relationship was also determined. Results on blends of asphalts having different compositions within the same grades revealed that both viscosity of the asphalt and composition as shown by the parameter $(N+A_1)/(P+A_2)$ were significant factors that affected the results of the pellet abrasion test. A linear mathematical relation was derived by which abrasion resistance can be estimated from the composition parameter and the viscosity of the asphalt.

The relation of asphalt composition to behavioral factors, other than embrittlement, as measured by the pellet abrasion tests, were not shown in these tests. But, data were obtained that can be related to on-going research by others on some of the asphalts included in the study discussed in this article.

Introduction

THIS ARTICLE is the fourth progress report of a comprehensive study of the physical and chemical properties of a series of asphalts produced for highway purposes. A total of 323 samples from 105 refineries were collected in 1954 and 1955 by the State highway departments and sent to the Bureau of Public Roads for study. The materials collected represented the greatest variety of asphalts used throughout the United States, as well as the greatest number of asphalts of the different consistency grades available and tested in any one investigation. The first two articles by the Public Roads Office of Research and Development Staff at the Herbert S. Fairbank Highway Research Station included the commonly determined test characteristics of the 85–100 penetration grade (*1*) [3] and the similar test characteristics of other grades—namely, 60–70, 70–85, 120–150, and 150–200 (*2*).

The third article by F. S. Rostler and R. M. White involved the determination of the composition and changes in composition of the 85–100 penetration grade asphalts (*3*). The theme of the investigation was based on the axiom that the properties of any material, including asphalt, are given by its chemical composition and physical state, and that changes in properties and behavior must parallel a corresponding change in chemical composition, and that it must be possible to predict properties and changes in properties from chemical composition.

The principal finding reported was that the parameter $(N+A_1)/(P+A_2)$ expressive of the ratio of the more reactive to the less reactive chemical compounds in the asphalt, permits rating of the 85–100 penetration grade asphalts in groups of equal durability when that property is measured as the ability to cement aggregate. Nine durability groups were first proposed. Later work showed that the groups should be reduced to five (*4*).

The research reported in this article was done under Bureau of Public Roads contract by the staff of Materials Research and Development, a division of Woodward, Clyde, Sherard and Associates. The article was written by representatives of the Bureau of Public Roads and the contractor. This research was undertaken primarily to extend the composition studies described in reference (*3*) to the other penetration grade asphalts and to determine the general validity of the parameter $(N+A_1)/(P+A_2)$ as a means of estimating the quality of an asphalt from the standpoint of retention of its cementing characteristics measured by the pellet abrasion test.

The objective of this article is to present a progress report and to make available to asphalt technologists the extensive collection of data obtained in the study, as well as conclusions reached to date.

Tests Made and Materials Tested

To gather sufficient data for a realistic analysis of the importance of chemical properties of asphalts, all of the 60–70 penetration grade and 120–150 penetration grade asphalts used in previous studies (*2*) were analyzed. A group of viscosity-graded asphalts also were included in this study. They had been collected by the Bureau of Public Roads for cooperative studies with the Asphalt Institute, State highway departments, and producers of asphalt. The precipitation method of chemical analysis, as described in 1962 by Rostler and White (*3*), was used. The constituents determined in the chemical analysis and the abbreviations used in this article, as well as in previous reports are:

A = *Asphaltenes:* Constituents insoluble in *n*-pentane.

N = *Nitrogen bases:* Constituents precipitated by 85-percent sulfuric acid. These are the most reactive components and include substantially all the nitrogen containing compounds.

A_1 = *First Acidaffins:* Constituents precipitated by 98-percent sulfuric acid after removal of the nitrogen bases. These are essentially unsaturated resinous hydrocarbons.

A_2 = *Second Acidaffins:* Constituents removed by fuming sulfuric acid (30-percent SO_3) after removal of nitrogen bases and first acidaffins. These are slightly unsaturated hydrocarbons.

P = *Paraffins:* Constituents that are nonreactive with fuming sulfuric acid (30-percent SO_3). These are saturated hydrocarbons.

The 1962 article and the author's closure published in the 1962 *Proceedings of The Association of Asphalt Paving Technologists* provide a general summary of the significance of the chemical analysis method and its advantages and disadvantages. Further

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[1] Presented by Mr. Halstead at the annual meeting of The Association of Asphalt Paving Technologists, Minneapolis, Minn., Feb. 1966.

[2] A division of Woodward, Clyde, Sherard and Associates.

[3] References indicated by italic numbers in parentheses are listed on p. 29.

Table 1.—60–70 penetration grade asphalts

Asphalt identification number	Asphalt composition, percent by weight					Paraffin fraction		Ratio,	Percent loss in pellet abrasion test at 77° F. on asphalt mixture ²			
	A	N	A_1	A_2	P	Wax indication ¹	Index of refraction, 77° F.	$\frac{N+A_1}{P+A_2}$	Original	Aged 3 days	Aged 7 days	Average of original and aged 7 days
120	29.9	22.1	23.1	17.7	7.2	+RT	1.4810	1.82	38	92	100(380)	69
121	32.3	16.3	23.3	20.1	8.0	+ice	1.4816	1.41	0.9	19	15	8
122	16.0	25.5	21.5	23.7	13.3	sl. RT	1.4826	1.27	2.8	11	14	8.4
123	27.2	16.9	20.8	21.5	13.6	−ice	1.4811	1.07	0.95	2.7	3.6	2.3
124	24.6	18.0	24.0	22.4	11.0	sl. RT	1.4827	1.26	3.6	35	35	19
125	24.2	14.8	23.2	22.7	15.1	sl. RT	1.4844	1.01	0.5	1.5	1.9	1.2
126	23.4	19.0	21.6	23.6	12.4	sl. ice	1.4817	1.13	0.3	1.8	4.6	2.5
127	29.6	22.8	25.1	16.4	6.1	+RT	1.4825	2.13	40	100(146)	100(391)	70
128	30.0	23.0	25.0	15.9	6.1	+RT	1.4818	2.18	30	83	100(344)	65
129	30.6	21.2	24.2	17.3	6.7	+RT	1.4811	1.89	12	58	84	48
130	23.4	22.6	20.4	22.1	11.5	sl. RT	1.4855	1.28	4.1	5.6	33	18
131	22.9	11.7	23.7	26.4	15.3	+RT	1.4838	0.85	0	0	0.1	0
132	22.1	22.7	22.0	21.9	11.3	sl. RT	1.4840	1.35	11	23	50	30
133	21.0	20.2	24.0	23.0	11.8	−ice	1.4830	1.27	3.1	1.7	3.8	3.4
134	19.1	15.0	26.7	25.2	14.0	sl. RT	1.4848	1.06	0.2	0.2	0.3	0.3
135	26.0	10.4	22.2	26.2	15.2	+RT	1.4817	0.79	0.7	1.0	0.5	0.6
136	31.0	20.4	24.7	17.3	6.6	+RT	1.4827	1.89	13	86	82	48
137	26.8	16.1	23.1	24.2	9.8	+RT	1.4819	1.15	14	41	58	36
138	22.2	23.0	21.7	22.2	10.9	sl. ice	1.4846	1.35	8	34	33	20.5
139	30.3	22.5	24.7	16.2	6.3	+RT	1.4831	2.10	39	96	94	67
140	27.4	14.0	25.6	24.2	8.8	+RT	1.4820	1.20	9	28	42	25
141	21.7	16.0	25.1	22.7	14.5	+RT	1.4869	1.11	0.3	0.2	0.2	0.3
142	18.1	19.9	28.2	23.3	10.5	sl. RT	1.4859	1.42	4.6	16	24	14
143	21.9	21.3	22.3	23.1	11.4	sl. ice	1.4836	1.26	4	7	26	15
144	21.2	23.8	21.7	23.9	9.4	+RT	1.4819	1.37	11	41	82	46
145	22.2	11.7	26.3	29.1	10.7	sl. RT	1.4837	0.96	1.0	2.2	2.8	1.9
146	23.4	21.3	21.6	21.8	11.9	+RT	1.4836	1.27	11	34	54	32
147	17.9	23.0	24.6	23.9	10.6	+RT	1.4855	1.38	26	53	78	52
148	21.8	20.1	23.8	22.3	12.0	−ice	1.4816	1.28	0.8	2.2	4.1	2.5
149	19.3	28.2	21.9	22.2	9.3	+RT	1.4847	1.56	7	33	55	31
150	20.0	15.6	23.6	26.9	13.9	+RT	1.4846	0.96	1.4	1.4	4.0	2.7
151	22.5	25.2	20.6	22.5	9.1	+RT	1.4824	1.45	13	57	76	44
152	15.0	26.4	24.7	23.3	10.6	+RT	1.4867	1.51	26	62	71	48
153	27.8	16.3	22.4	24.3	9.2	+RT	1.4811	1.16	7	15	29	18
154	20.2	20.8	23.8	26.1	9.1	+RT	1.4858	1.27	42	93	100(261)	71
155	20.0	27.6	19.8	23.1	9.5	sl. RT	1.4832	1.45	1.0	5	20	10
156	28.5	10.3	24.3	26.7	10.2	+RT	1.4816	0.94	1.8	3.2	3.2	2.5
157	5.0	25.5	32.1	25.5	11.9	sl. RT	1.4877	1.54	25	10	45	35
158	21.8	11.4	27.6	28.6	10.6	+RT	1.4826	1.00	1.4	2.1	6.0	3.7
159	28.5	19.4	14.7	20.9	16.5	sl. RT	1.4803	0.91	0.07	0.08	0.3	0
160	24.4	11.5	25.0	27.4	11.7	sl. RT	1.4798	0.93	0.12	0.1	0.08	0
161	31.4	10.8	20.0	24.3	13.5	sl. RT	1.4774	0.82	0.05	0.1	0.6	0
162	20.1	15.3	23.5	25.8	15.3	sl. RT	1.4846	0.94	1.0	0.29	0.90	1
163	31.1	11.6	19.9	24.0	13.4	+RT	1.4786	0.85	0.40	0.43	1.9	1
164	27.1	15.1	23.6	24.2	10.0	+RT	1.4822	1.13	8.0	10.0	46	27
165	24.2	21.3	16.0	23.4	15.1	+RT	1.4846	0.97	0.40	1.7	0.50	0.5
166	22.6	13.3	28.1	26.6	9.4	sl. RT	1.4815	1.17	1.75	0.24	5.4	3.6

[1] +RT represents the presence of wax at room temperature.
sl. RT represents the presence of a slight amount of wax at room temperature.
+ice represents the presence of wax when temperature was lowered by use of crushed ice.
−ice indicates that no wax was present in the paraffin fraction when temperature was lowered by use of crushed ice.
sl. ice indicates presence of a slight amount of wax when temperature was lowered by use of crushed ice.

[2] Figures in parentheses are number of revolutions to 100 percent loss.

review of the principles involved also has been published in the proceedings of the quality control conference, part II, sponsored by the Bureau of Public Roads in 1965 (*4*). In reference (*4*) details are given on the characteristics of the individual fractional components. This article is essentially a continuation of the work reported in 1962, therefore, reference should be made to the published discussion for a better understanding of the principles involved.

In addition to the Rostler-Sternberg analysis, tests were made for qualitatively determining the presence of wax in the paraffin fraction, the index of refraction of the paraffins, and the pellet abrasion test at 77° F. for specimens after mixing (6 minutes at 325° F.) and after 3 days and 7 days of aging at 140° F. Data from these tests are given in tables 1 through 3. The ranges of chemical compositions for each of the penetration grades are shown in table 4. Data for the 85–100 penetration grade previously reported in reference (*3*) are included. The content of any fractional component within each grade varied widely and, because of several interacting factors, specific relations cannot be determined by using data for single components. However, previous research had demonstrated that the parameter $(N+A_1)/(P+A_2)$, expressive of the ratio of more to less chemically reactive components, controls to a considerable degree the retention by an asphalt of its cementing quality as measured by the pellet abrasion test.

In figures 1 through 3, the average abrasion loss at 77° F. has been plotted against the ratio, $(N+A_1)/(P+A_2)$. The average of the abrasion loss immediately after mixing and after 7 days of accelerated aging was chosen as a measure of cementing quality of the asphalt. This average was chosen as it represents experimentally both the original cementing effect and the retained cementing power. The dotted line on figures 1 through 3 indicates the position of the curve for the 85–100 grade asphalts as published by Rostler and White in 1962 (*3*). For the 60–70 grade asphalts, scatter of the data is indicated in the range of parameters between 1.0 and 1.6, but a trend similar to that previously indicated for the 85–100 grade asphalts is shown. For the 120–150 grade asphalts the same trend is apparent but the average abrasion loss is much less for equal values of the parameter than indicated by the other grades. The data in figure 3 for the viscosity graded asphalts show a shot-gun pattern. This lack of trend indicates that grading an asphalt by consistency measured at one temperature and rating its quality by performance measured at another is not a fruitful approach. Asphalts graded by viscosity at 140° F., because of large variations in temperature susceptibility, can have entirely different relative consistencies at 77° F., at which temperature the abrasion test was run. When the same series of asphalts is subdivided into groups according to penetration or viscosity at 77° F., the viscosity graded asphalts show the same trend as the specimens of the penetration graded asphalts.

The difference shown by the various penetration and viscosity grades of asphalts and the spread of the data within each group indicate a complex relation that is affected by other factors. One of these is obviously the consistency, or viscosity, of the asphalt. This relation will be discussed later in more detail.

One of the major objectives of the study reported here was to evaluate the influence of the ratio of the more reactive to less reactive maltenes constituents $(N+A_1)/(P+A_2)$ on the cementing quality of the asphalt measured by the pellet abrasion test over a wide range of consistencies. Because of the many tests that would be necessary to test all asphalts at several temperatures, tests were considered desirable on a limited number of typical materials that would not be related to a particular geographic origin. To accomplish this objective, composite blends were prepared of the materials within each penetration grade. Compositing was done on the basis of five groupings, whose composition parameters were:

Group I, Minimum (0.54) to 1.00
Group II, 1.01 to 1.20
Group III, 1.21 to 1.50
Group IV, 1.51 to 1.70
Group V, 1.71 to maximum (2.24)

Group I

The BPR numbered asphalts of the different penetration grades used in the blends for Group I were:

60–70 penetration grade (12 asphalts): 131, 135, 145, 150, 156, 158, 159, 160, 161, 162, 163, and 165.

85–100 penetration grade (49 asphalts): 10, 11, 16, 18, 19, 21, 22, 23, 26, 29, 30, 31, 32, 33, 34, 35, 37, 41, 42, 43, 45, 46, 47, 48, 49, 51, 53, 55, 56, 57, 58, 60, 61, 62, 63, 64, 65, 66, 67, 68, 72, 73, 75, 76, 77, 78, 79, 80, and 83.

120–150 penetration grade (16 asphalts): 204, 209, 215, 225, 228, 230, 231, 232, 233, 234, 235, 239, 240, 242, 244, and 245.

Table 2.—120–150 penetration grade asphalts

Asphalt identification number	Asphalt composition, percent by weight					Paraffin fraction		Ratio, $\frac{N+A_1}{P+A_2}$	Percent loss in pellet abrasion test at 77° F. on asphalt mixture —			
	A	N	A_1	A_2	P	Wax indication[1]	Index of refraction, 77° F.		Original	Aged 3 days	Aged 7 days	Average of original and aged 7 days
196	28.1	17.8	23.5	21.1	9.5	−ice	1.4806	1.35	0.35	0.32	0.60	0.5
197	26.0	24.4	25.6	16.8	7.2	+RT	1.4791	2.08	2.0	43	55	28
198	12.9	25.1	22.1	23.9	16.0	sl. RT	1.4823	1.18	0.3	0.3	0.6	0.4
199	19.2	21.1	25.6	24.0	10.1	+RT	1.4808	1.37	4.0	4.0	24	14
200	26.6	23.4	25.8	16.6	7.6	+RT	1.4771	2.03	0.9	7	55	28
201	25.7	27.4	21.4	16.3	9.2	+RT	1.4819	1.91	4.1	17	51	28
202	27.2	22.4	24.9	17.7	7.8	+RT	1.4780	1.85	1.7	25	41	21
203	19.5	23.2	21.2	23.3	12.8	sl. ice	1.4838	1.23	0.4	0.7	13	6.7
204	18.9	12.6	25.6	27.1	15.8	+RT	1.4822	0.89	0.1	0.03	0	0
205	20.4	18.0	22.3	24.8	14.5	sl. ice	1.4815	1.03	0.1	0.1	0.2	0.1
206	19.0	19.3	24.1	25.0	12.6	−ice	1.4824	1.15	0.12	1.7	0.20	0.2
207	27.6	22.7	25.5	17.1	7.1	+RT	1.4762	1.99	1.90	20	55	28
208	16.1	20.4	24.1	24.6	14.8	+RT	1.4828	1.13	0.05	0.05	0.12	0.1
209	18.4	23.5	16.7	27.5	13.9	+RT	1.4827	0.97	0.08	0.08	0.22	0.10
210	20.7	21.9	22.9	23.7	10.8	+RT	1.4788	1.30	0.7	8	7	3.8
211	19.2	22.7	20.5	24.5	13.1	+RT	1.4804	1.15	0.20	2	2	1.1
212	18.2	17.2	23.8	25.4	15.4	+RT	1.4822	1.01	0	0	0.1	0
213	15.5	23.6	22.6	25.8	12.5	+RT	1.4815	1.21	2.1	3.9	9	5.5
214	14.4	23.6	25.0	26.2	10.8	+RT	1.4841	1.31	3.5	11	20	12
215	17.6	15.8	23.2	28.0	15.4	+RT	1.4815	0.90	0.12	0	0.05	0.1
216	15.9	21.1	23.1	24.8	15.1	+RT	1.4820	1.11	0	0.12	0.05	0
217	21.8	24.0	22.1	21.6	10.5	+RT	1.4781	1.44	0.60	2.7	17	8.8
218	20.5	19.3	22.6	26.4	11.2	+RT	1.4793	1.11	1.3	1.7	3.8	2.5
219	19.6	19.6	22.0	26.8	12.0	+RT	1.4798	1.07	0.40	0.15	1.07	0.74
220	19.7	23.7	22.4	23.6	10.6	+RT	1.4786	1.35	1.1	9	15	8.0
221	20.4	25.6	23.1	21.3	9.6	+RT	1.4791	1.58	5.5	31	50	28
222	17.7	25.1	21.1	24.0	12.1	+RT	1.4814	1.28	2.8	9	20	11
223	21.7	24.7	22.4	21.0	10.2	+RT	1.4784	1.51	1.3	8	31	16
224	24.2	14.9	24.0	25.8	11.1	+RT	1.4785	1.05	0.22	0.15	2.9	1.6
225	19.5	19.0	20.8	23.9	16.8	+RT	1.4807	0.98	0.02	0.08	0.80	0.4
226	17.4	19.5	23.8	28.7	10.6	+RT	1.4815	1.10	1.9	10	48	25
227	15.6	29.9	19.4	24.6	10.5	sl. RT	1.4822	1.41	0.47	0.50	3.0	1.7
228	25.7	10.3	23.5	28.1	12.4	+RT	1.4769	0.83	0.12	0.05	0.10	0.10
229	5.0	18.2	38.4	26.0	12.4	+RT	1.4854	1.47	0.20	0.34	1.3	0.7
230	19.2	12.5	26.3	29.7	12.3	+RT	1.4797	0.92	0	0.08	0.30	0.15
231	24.1	21.4	15.4	22.3	16.8	+ice	1.4805	0.94	0.08	0.02	0.02	0
232	19.1	13.3	25.6	30.6	11.4	+RT	1.4799	0.93	0.02	0.05	0.20	0.10
233	29.5	10.5	19.8	25.3	14.9	+RT	1.4770	0.75	0	0.2	0.2	0.10
234	19.8	12.9	21.6	26.8	18.9	+RT	1.4802	0.76	0.02	0.10	0.05	0
235	28.1	10.6	20.2	26.5	14.6	+RT	1.4764	0.75	0	0.02	0.08	0
236	6.1	51.2	51.2	35.2	7.5	+RT	1.4822	1.20	0.12	0.08	0.49	0.30
237	20.3	16.9	27.6	19.2	16.0	+RT	1.4930	1.26	1.9	14	25	13
238	21.4	15.1	25.3	27.0	11.2	sl. RT	1.4801	1.06	1.8	1.1	4.8	3.2
239	13.9	19.5	15.2	29.0	22.4	−ice	1.4970	0.67	0.84	1.3	1.3	1.1
240	18.2	17.2	21.7	25.0	17.9	+RT	1.4803	0.91	0.12	0	0.05	0.08
241	13.0	26.5	28.7	23.7	8.1	sl. RT	1.4846	1.74	0.59	2.0	5.4	3.0
242	20.5	22.0	15.7	26.2	15.6	+RT	1.4843	0.90	0.22	0.08	0.34	0.28
243	15.1	17.5	29.1	26.0	12.3	+RT	1.4827	1.22	0.15	0.10	0.10	0.12
244	18.6	15.1	25.4	30.8	10.1	+RT	1.4806	0.99	0.10	0.08	0.08	0.10
245	21.9	13.2	23.7	29.6	11.6	+RT	1.4790	0.90	0.15	0.15	0.19	0.17
246	27.8	27.0	20.9	12.6	11.7	+RT	1.4792	1.97	1.2	21	51	26
247	8.1	39.6	16.3	22.8	13.2	−ice.	1.4860	1.55	1.7	5.6	4.7	3.2
248	18.0	37.0	16.5	16.0	12.5	+RT	1.4843	1.88	15	49	56	35.5
249	9.8	38.9	15.2	21.8	14.3	−ice	1.4857	1.50	3	13	32	17.5
250	29.4	26.4	22.4	12.4	9.4	+RT	1.4770	2.24	2.0	22	54	28
251	29.1	28.4	20.6	12.5	9.4	+RT	1.4775	2.24	2.3	44	54	28

[1] +RT represents the presence of wax at room temperature.
sl. RT represents the presence of a slight amount of wax at room temperature.
−ice indicates that no wax was present in the paraffin fraction when temperature was lowered by use of crushed ice.
sl. ice indicates presence of a slight amount of wax when temperature was lowered by use of crushed ice.

Table 3.—Asphalts graded by viscosity at 140° F.

Asphalt code number	Asphalt composition, percent by weight					Paraffin fraction		Ratio, $\frac{N+A_1}{P+A_2}$	Percent loss in pellet abrasion test at 77° F. on asphalt mixture			
	A	N	A_1	A_2	P	Wax indication	Index of refraction, 77° F.		Original	Aged 3 days	Aged 7 days	Average of original and aged 7 days
A-5	19.0	19.0	21.7	27.2	13.1	+RT	1.4827	1.01	0.1	1.5	2.8	1.5
A-10	20.5	19.4	21.6	25.5	13.0	+RT	1.4826	1.07	2.2	3.1	15	8.6
A-20	21.6	20.1	21.7	23.8	12.8	+RT	1.4817	1.14	3.2	24	44	23.6
B-5	14.0	18.6	28.0	26.3	13.1	+RT	1.4839	1.18	0.2	0.25	0.4	0.3
B-10	15.5	18.0	29.4	24.8	12.3	+RT	1.4837	1.28	1.2	1.9	5	3.1
B-20	18.1	18.5	28.6	22.8	12.0	+RT	1.4844	1.35	7	43	49	28
C-5	25.2	10.6	23.9	26.4	13.9	+RT	1.4766	0.86	0	0	0.2	0.1
C-10	27.4	13.1	24.1	24.4	11.0	+RT	1.4775	1.05	0.05	0.5	1.8	0.9
C-20	27.9	12.0	24.4	25.6	10.1	+RT	1.4775	1.02	3.0	6	19	11
D-5	19.3	19.0	21.0	25.4	15.3	sl. RT	1.4815	0.98	0.02	0.32	0.08	0.05
D-10	20.2	17.7	23.5	24.6	14.0	sl. ice	1.4819	1.07	0.10	0.10	0.24	0.17
D-20	21.6	19.4	22.3	24.2	12.5	sl. RT	1.4822	1.14	1.5	1.7	1.1	1.3
E-5	16.6	13.0	30.8	23.4	16.2	+RT	1.4839	1.11	7	7	9	8
E-10	16.5	13.6	33.0	22.6	14.3	+RT	1.4851	1.26	61	63	87	74
F-5	14.3	19.7	25.4	25.8	14.8	+RT	1.4836	1.11	0.22	0.02	0.17	0.20
F-10	17.8	16.7	26.2	24.3	15.0	+RT	1.4833	1.09	0.25	0.08	0.24	0.24
F-20	20.1	13.0	27.8	24.3	14.8	+RT	1.4813	1.04	0.70	0.02	0.20	0.45
G-5	19.1	14.4	26.5	27.4	12.6	+RT	1.4805	1.02	0.12	0.15	0.98	0.55
G-10	20.8	15.2	26.0	26.7	11.3	+RT	1.4810	1.08	1.3	2.4	6.2	3.8
G-20	21.6	15.9	27.1	25.2	10.2	+RT	1.4814	1.21	5.6	19	42	24
H-5	19.5	16.8	21.3	28.2	14.2	+RT	1.4806	0.90	0.34	0.32	0.84	0.59
H-10	21.2	18.4	20.0	26.8	13.6	+RT	1.4806	0.95	0.6	1.1	6.0	3.3
H-20	23.2	19.7	19.9	25.2	11.9	+RT	1.4812	1.07	10	52	52	31
I-5	14.8	16.4	26.9	31.1	10.8	+RT	1.4825	1.03	0.30	0.4	1.3	0.80
I-10	16.7	17.1	26.9	29.6	9.7	+RT	1.4827	1.12	5.2	22	28	17
I-20	18.3	17.2	26.7	29.0	8.8	+RT	1.4828	1.16	62	91	93	78
J-5	24.5	23.3	24.5	17.8	9.9	+RT	1.4770	1.73	0.10	0.12	18	9.0
J-10	26.1	24.5	24.4	16.9	8.1	+RT	1.4769	1.96	0.80	12	53	27
J-20	27.6	24.4	24.4	16.3	7.3	+RT	1.4773	2.07	12	43	74	43
J-40	28.9	25.0	24.1	15.5	6.5	+RT	1.4786	2.23	32	79	98	65
K-5	19.5	18.3	23.3	18.4	20.5	+RT	1.4790	1.07	1.0	0.4	2.1	1.5
K-10	19.2	20.3	25.4	18.5	16.6	+RT	1.4805	1.30	29	24	55	42
K-20	19.2	21.0	27.3	18.3	14.2	+RT	1.4806	1.49	83	80	100(385)	92
K-40	18.7	22.7	29.4	16.5	12.7	+RT	1.4826	1.78	100	100(197)	100(114)	100
L-5	22.4	13.3	23.4	27.4	13.5	+RT	1.4790	0.90	0.10	0.10	0.12	0.11
L-10	23.6	13.7	24.6	25.8	12.3	+RT	1.4791	1.01	0.70	1.6	18	9.4
L-20	25.3	14.6	24.1	25.0	11.0	+RT	1.4800	1.08	4.4	8	28	16
L-40	26.7	15.0	24.2	24.3	9.8	+RT	1.4810	1.15	25	60	87	56
M-5	17.3	30.8	15.3	20.3	16.3	sl. RT	1.4851	1.26	0.60	1.6	7	3.8
M-10	18.6	30.4	15.4	20.4	15.2	sl. RT	1.4850	1.29	11	23	39	25
M-20	19.7	31.5	14.5	19.9	14.4	+RT	1.4842	1.34	21	27	44	32.5
N-5	18.1	34.8	14.1	18.0	15.0	−ice	1.4890	1.48	1.0	1.3	7	4.0
N-10	18.6	35.2	15.1	17.2	13.9	−ice	1.4897	1.62	3.2	12	19	26
N-20	19.9	37.2	13.5	16.7	12.7	−ice	1.4904	1.72	30	53	100(355)	65
O-5	9.7	38.7	16.3	20.2	15.1	−ice	1.4863	1.56	1.1	2.0	14	7.6
O-10	11.0	40.7	15.7	19.9	12.9	−ice	1.4867	1.71	19	37	60	39
O-20	12.2	42.1	15.8	18.1	11.6	−ice	1.4868	1.93	70	90	100(312)	85
O-40	13.5	42.9	15.1	18.2	10.3	−ice	1.4872	2.04	100(390)	100(185)	100(94)	100

+RT represents the presence of wax at room temperature.
sl. RT represents the presence of a slight amount of wax at room temperature.
−ice indicates that no wax was present in the paraffin fraction when temperature was lowered by use of crushed ice.
sl. ice indicates presence of a slight amount of wax when temperature was lowered by use of crushed ice.
Figures in parentheses are number of revolutions to 100 percent loss.

Group II

The BPR numbered asphalts of the different penetration grades used in the blends for Group II were:

60–70 penetration grade (10 asphalts): 123, 125, 126, 134, 137, 140, 141, 153, 164, and 166.

85–100 penetration grade (16 asphalts): 3, 6, 7, 8, 9, 12, 17, 20, 27, 28, 38, 39, 50, 59, 113, and 117.

120–150 penetration grade (11 asphalts): 198, 205, 206, 208, 211, 212, 216, 218, 219, 224, and 238.

Group III

The BPR numbered asphalts of the different penetration grades used in the blends for Group III were:

60–70 penetration grade (14 asphalts): 121, 122, 124, 130, 132, 133, 138, 142, 143, 144, 146, 147, 148, and 151.

85–100 penetration grade (11 asphalts): 1, 15, 25, 52, 71, 93, 104, 105, 110, 114, and 118.

120–150 penetration grade (14 asphalts): 196, 199, 203, 210, 213, 214, 217, 220, 222, 227, 229, 237, 243, and 249.

Group IV

The BPR numbered asphalts of the different penetration grades used in the blends for Group IV were:

60–70 penetration grade (3 asphalts): 149, 152, and 157.

85–100 penetration grade (6 asphalts): 5, 14, 24, 91, 106, and 108.

120–150 penetration grade (3 asphalts): 221, 223, and 247.

Group V

The BPR numbered asphalts of the different penetration grades used in the blends for Group V were:

60–70 penetration grade (6 asphalts): 120, 127, 128, 129, 136, and 139.

85–100 penetration grade (6 asphalts): 90, 92, 94, 95, 96, and 99.

120–150 penetration grade (10 asphalts): 197, 200, 201, 202, 207, 241, 246, 248, 250, and 251.

As typical behavior was desi[illegible] those asphalts that had shown anomalo[illegible] behavio[illegible] in the previous research of Rostle[illegible] Whi[illegible]

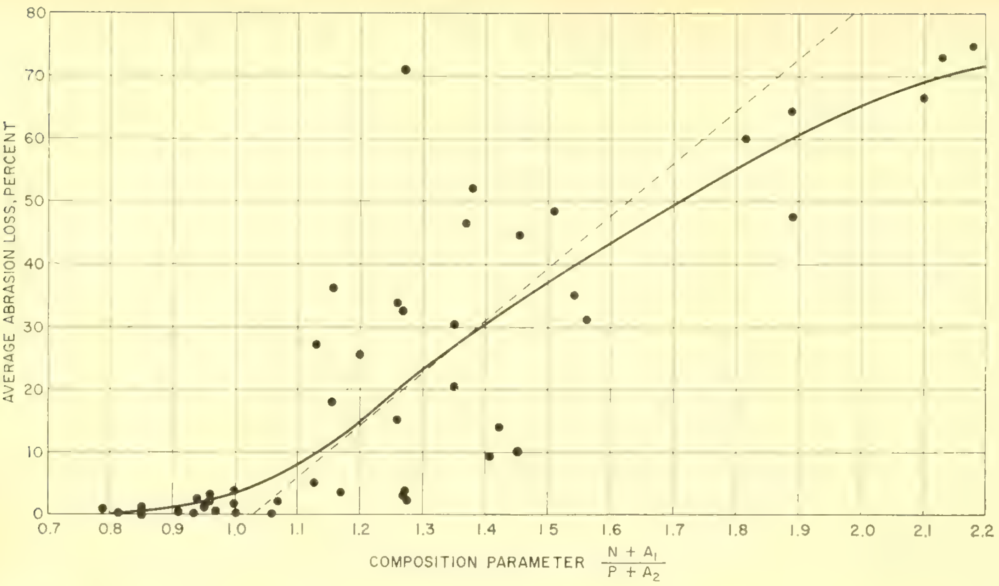

Figure 1.—Relation of average abrasion loss to composition parameter for 60–70 penetration grade asphalts.

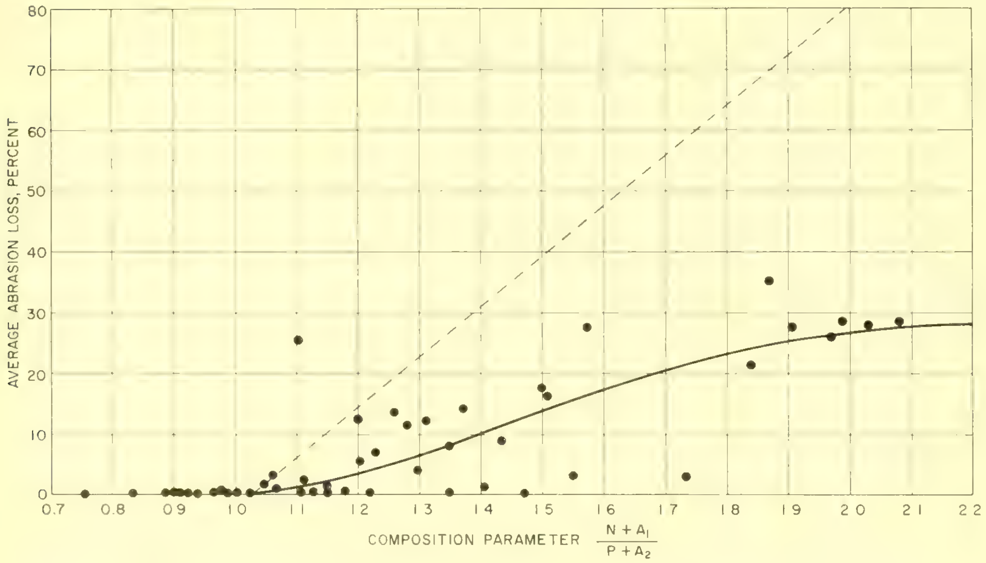

Figure 2.—Relation of average abrasion loss to composition parameter for 120–150 penetration grade asphalts.

Table 4.—Smallest and largest amount of each component present in individual asphalts

	Asphalt number and smallest amount		Asphalt number and largest amount	
60–70 PENETRATION GRADE				
	No.	*Amt.*	*No.*	*Amt.*
Components:				
A	157	5.0	121	32.3
N	156	10.3	149	28.2
A_1	159	14.7	157	32.1
A_2	128	15.9	145	29.1
P	127 and 128	6.1	159	16.5
$\frac{N + A_1}{P + A_2}$	135	0.79	128	2.18
85–100 PENETRATION GRADE				
Components:				
A	93	11.4	103	35.9
N	37	6.6	96	41.9
A_1	97	13.1	71	28.3
A_2	103	12.4	60	33.0
P	13	6.8	84	23.6
$\frac{N + A_1}{P + A_2}$	84	0.54	100	2.06
120–150 PENETRATION GRADE				
Components:				
A	229	5.0	233	29.5
N	228	10.3	247	39.6
A_1	239 and 249	15.2	229	38.4
A_2	250	12.4	244	30.8
P	207	7.1	239	22.4
$\frac{N + A_1}{P + A_2}$	239	0.67	250 and 251	2.24
ASPHALTS GRADED BY VISCOSITY AT 140° F.				
Components:				
A	O–5	9.7	J–40	28.9
N	C–5	10.6	O–40	42.9
A_1	N–20	13.5	E–10	33.0
A_2	J–40	15.5	I–5	31.1
P	J–40	6.5	K 5	20.5
$\frac{N + A_1}{P + A_2}$	C–5	0.86	J–40	2.23

(*3*) were not included in the composite blends. Also, because of the large number of 85–100 grade asphalts that fell in Group I, some of these asphalts were not included in this composite. The groupings listed in the preceding paragraphs were selected after preliminary studies of blends of the 85–100 penetration grade asphalts. Blends made on the basis of durability as defined in reference (*3*) gave a reasonable relation between composition and abrasion resistance. However, these preliminary results indicated that the relative differences in asphalts measured by the pellet abrasion test did not justify nine groupings. It was also decided that groupings by chemical composition provided a more precise classification and was the more direct approach to testing the influence of composition on performance. On the basis of previous study of data in which composition was related to durability measured as loss of cementing power, it was postulated that the abrasion resistance would decrease as the group number increased.

Precipitation Analysis

One of the major advantages of the precipitation method of analysis is that the results are contingent on the reactivity of the component groups, and independent of interactions or equilibrium considerations between different components. Good evidence supporting this fact is shown by the data given in tables 5, 6, and 7. These tables compare the analytically determined composition of the composites for each penetration grade with the average calculated from the data on the individual asphalts included in the composites. The measured penetrations at 77° F. (tables 5, 6, and 7) for a number of the blends are slightly outside the limits of the grade represented. This difference was caused by heating the samples during the blending process, as well as the fact that the blends were made with material remaining after most of the other tests had been completed. As the purpose of these tests was to compare relative behavior, the deviations from grade are of no consequence.

The original chemical properties of the blends for each group within the 60–70, 85–100, and 120–150 asphalt penetration grades are shown in table 8. This table also shows the same properties for the thin-film residues and for asphalt recovered from the abrasion test mixture immediately after mixing and after 7 days of aging. The chemical properties shown include data on a qualitative test for wax and the index of refraction for the paraffins. The ratios of highly reactive to less reactive components $(N+A_1)/(P+A_2)$ are also given.

Fractional Composition

Figure 4 depicts, in the form of two sets of bar graphs, the results of the fractional analysis of the composite blends of the three penetration grades of asphalt. The lower set of bars shows the asphaltenes content as

Table 5.—Comparison of test result and calculated result for percentage of components in blends of 60–70 penetration grade asphalts

	Group I, blend of 12 asphalts		Group II, blend of 10 asphalts		Group III, blend of 14 asphalts		Group IV, blend of 3 asphalts		Group V, blend of 6 asphalts	
	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result
Components:										
A ---- pct. by wt.	24.8	25.2	24.2	24.7	21.8	22.0	14.9	13.1	27.9	30.2
N ---- do	14.4	13.4	17.0	15.7	23.0	21.7	26.5	26.7	24.7	22.0
A_1 ---- do	21.1	22.2	22.8	24.0	21.1	22.8	24.9	25.9	23.5	24.5
A_2 ---- do	26.1	25.8	24.2	23.9	23.2	22.6	23.3	23.7	17.2	16.8
P ---- do	13.6	13.4	11.8	11.7	10.9	10.9	10.4	10.6	6.7	6.5
$\frac{N+A_1}{P+A_2}$	0.89	0.91	1.11	1.12	1.29	1.33	1.53	1.53	2.02	2.00
Penetration of original	58	61	58	60	60	61	67	64	61	63
Thin-film oven test [1] pct. loss	0.14	0.08	0.09	0.05	0.11	0.05	+0.01	0.00	0.38	0.27
Residue:										
Penetration at 77° F.	43	43	41	41	41	40	47	44	40	40
Percent of original penetration	74	70	71	68	68	66	70	69	66	63
Percent loss in pellet abrasion test at 77° F. on—										
Original mix	0.35	0.7	1.1	4.2	3.6	7.4	17	19	33	29
Mix aged 3 days	0.5	0.9	1.1	10.1	9	22	49	45	74	86
Mix aged 7 days	2.4	1.7	13	19.1	45	40	62	57	96	93
Average loss on original mix and mix aged 7 days	1.4	1.4	7.1	11.7	24	24	39.5	38	64.5	61

[1] Residue from thin-film oven test ⅛-in. film, 5 hours at 325° F.

Table 6.—Comparison of test result and calculated result for percentage of components in blends of 85–100 penetration grade asphalts

	Group I, blend of 19 asphalts		Group II, blend of 16 asphalts		Group III, blend of 11 asphalts		Group IV, blend of 6 asphalts		Group V, blend of 6 asphalts	
	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result
Components:										
A ---- pct. by wt.	22.6	23.3	20.4	21.1	20.6	22.4	24.7	23.7	24.7	25.4
N ---- do	15.4	13.0	22.1	18.4	24.9	22.1	25.5	24.6	31.4	28.1
A_1 ---- do	20.5	21.6	21.0	22.7	21.4	22.1	21.3	22.2	17.6	20.0
A_2 ---- do	27.5	28.3	24.8	26.1	23.5	23.9	20.2	20.6	16.2	16.5
P ---- do	14.0	13.8	11.7	11.7	9.6	9.5	8.3	8.9	10.1	10.0
$\frac{N+A_1}{P+A_2}$	0.87	0.82	1.18	1.09	1.40	1.32	1.64	1.59	1.86	1.82
Penetration of original	78	89	84	90	84	89	81	91	81	90
Thin-film oven test [1] pct. loss	0.13	0.04	0.22	0.08	0.19	0.04	0.44	0.08	0.62	0.11
Residue:										
Penetration at 77° F.	55	56	56	55	52	52	48	54	52	51
Percent of original penetration	70	63	67	61	62	58	59	59	64	57
Percent loss in pellet abrasion test at 77° F. on—										
Original mix	0.3	1.6	0.7	2.4	10.0	9.4	22	9.5	29	22
Mix aged 7 days	0.9	5.1	6.0	8.9	53.0	38.5	69	81.2	96	99
Average loss on original mix and mix aged 7 days	0.6	3.3	3.4	5.6	31.5	24	45.5	45.4	62.5	60.5

[1] Residue from thin-film oven test ⅛-in. film, 5 hours at 325° F.

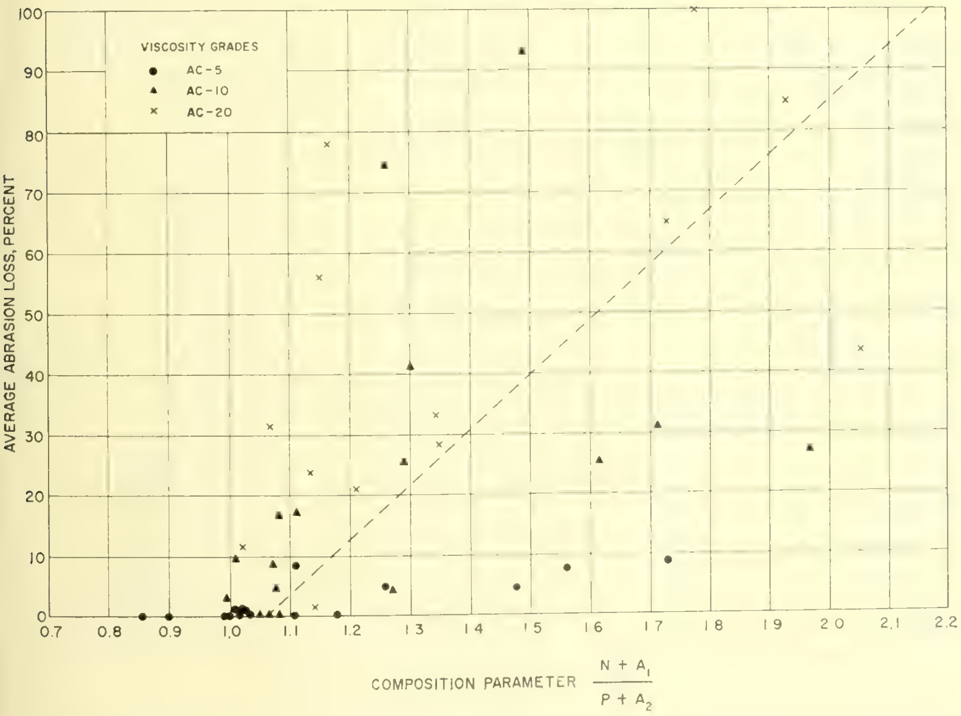

Figure 3.—Relation of average abrasion loss to composition parameter for viscosity graded asphalts.

percent of asphalt. As shown, asphaltenes content varies without definite pattern from blend to blend. This is true because the amount of asphaltenes present in an asphalt is governed primarily by the amount and the viscosity of the other components, which depends on manufacturing procedures aimed at producing an asphalt of the specified consistency.

Because of the variation in asphaltenes content, the balance of the components are graphed in the upper set of bars as a percent of maltenes—total asphalt minus asphaltenes—rather than as percent of the total asphalt. This plot of the data shows that in all three penetration grades the nitrogen bases increased significantly from Group I to Group V; whereas, first acidaffins were relatively constant. The increase in the $(N + A_1)$ portion from group to group was therefore primarily an increase in nitrogen bases. Both second acidaffins (A_2) and paraffins (P) decreased from Group I to Group V, so the decrease in the $(P + A_2)$ portion from group to group was the result of a decrease in both constituents.

The bars in figure 5 illustrate the changes in chemical composition during mixing and during aging for 7 days. Although no attempt was made to determine definitely how much of the changes was caused by volatility and how much by chemical reaction during mixing or aging, the close agreement between the compositions of the residues from the thin-film oven test and the residue after mixing, as shown by data in table 8 indicates that much of the change was caused by chemical reaction. This agreement of results is also indicative that the conditions chosen for the mixing in the pellet abrasion test, 6 minutes at 325° F., approximate the effect of mixing in commercial hot mix plants.

For most asphalt blends the amounts of constituents, other than asphaltenes, either decreased or underwent no significant change during mixing and aging. The increases in asphaltenes therefore approximated the sum of changes in all other constituents. Generally the largest decreases were in the first acidaffins. Usually only small changes occurred in the second acidaffins fraction, and paraffins essentially were unchanged for all blends. The content of nitrogen bases increased for some blends and decreased for others. This suggests that some of the reaction products from changes in other constituents are reactive with 85-percent sulfuric acid. Thus, the analytical result would be the net effect of two opposite changes. The general trend shown is for the total changes in composition to increase as the group number increases. The most significant change being the decreases in the sums of nitrogen bases and first acidaffins. However, the blends of the 85–100 grade showed almost uniform changes for all groups.

Pellet Abrasion Test

The pellet abrasion test, described previously (*3*), was used to measure the relative cementing quality of the asphalts. Test temperature was more exactly controlled in the study reported here. Also, tests were made over a range of temperatures from 40° F. to 90° F. A detailed description of the pellet abrasion test and the apparatus used is given in a paragraph near the end of this article.

Results of the abrasion tests, over the range of temperatures from 40° F. to 90° F. are given in table 9 for each of the three penetration grade blends. Data are included on tests made immediately after mixing, after 3 days aging of the mixture at 140° F., and after 7 days of aging at 140° F. The test results indicate the influence of consistency of the asphalt on the abrasion test results. Abrasion loss varied from 0 to 100 over a relatively narrow temperature range. Figure 6 illustrates the average abrasion loss for the unaged specimens and the specimens after 7 days of aging plotted against the ratio $(N+A_1)/(P+A_2)$ for each of the asphalt penetration grades at each temperature. Because of the sensitivity of asphalt consistency to temperature, only the pellet abrasion tests at 65° F. and 77° F. gave sufficient data between 0 and 100 percent to be useful. Therefore, the results at 77° F. were used for further evaluation. Also 77° F. was used as the standard temperature for the abrasion evaluation for continuity with the previous work in which this temperature was used.

Because of the significant dependence of abrasion resistance on the viscosity of the asphalt, the viscosities of the blends at different temperatures were determined for a range of shear rates. The viscosity data are given in table 10. Figure 7 shows the square root of the average abrasion loss at 77° F. plotted against the viscosity of the residues from the thin-film oven test loss at a shear rate of 0.01 sec.$^{-1}$ The trial and error method was used to determine that the square root of the abrasion data gave better indication of relative abrasion resistance than did the percentage loss. Also because the characteristics of the thin-film residue are known to approximate closely the characteristics of the asphalts in the abrasion specimen prior to aging, the thin-film residue viscosities provide a more accurate comparison than do the viscosities of the original blends. However, the general trends indicated by data in figure 7 also are shown by figure 8 in which the abrasion data are plotted against viscosities of the original blends. The significant difference between data is a greater separation of the curves for Group IV and V asphalts in figure 8 than in figure 7.

The relations indicated by data in figures 7 and 8 demonstrate that for asphalts of equal viscosities the ratio $(N + A_1)/(P + A_2)$ has a significant effect on the cementing quality of an asphalt when the asphaltenes are relatively constant. The effects of variations in the

Table 7.—Comparison of test result and calculated result for percentage of components in blends of 120–150 penetration grade asphalts

	Group I, blend of 16 asphalts		Group II, blend of 11 asphalts		Group III, blend of 14 asphalts		Group IV, blend of 3 asphalts		Group V, blend of 10 asphalts	
	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result	Test result	Calculated result
Composition										
A ... pct. by wt	21.2	20.8	19.3	18.8	17.9	17.3	16.9	16.7	24.8	25.0
N ... do	16.1	15.6	20.0	19.3	24.8	23.3	31.3	30.0	28.4	26.6
A_1 do	20.8	21.3	21.6	23.1	22.1	24.0	19.2	20.6	21.4	23.2
A_2 do	26.8	27.3	25.7	25.4	23.3	23.6	21.4	21.7	16.3	16.2
P ... do	15.1	15.0	13.4	13.4	11.9	11.8	11.2	11.0	9.1	9.0
$\frac{N+A_1}{P+A_2}$	0.88	0.87	1.06	1.09	1.33	1.33	1.55	1.55	1.96	1.98
Penetration of original	133	130	139	137	137	130	131	127	127	132
Thin-film oven test[1] pct. loss	0.20	0.13	0.09	0.05	0.27	0.20	0.31	0.21	0.74	0.81
Residue:										
Penetration at 77° F	84	79	85	82	83	76	79	78	77	67
Percent of original penetration	63	61	61	60	61	58	60	61	61	51
Percent loss in pellet abrasion test at 77° F. on –										
Original mix	0.15	0.12	0.1	0.1	0.2	1.5	0.9	2.8	0.9	3.2
Mix aged 3 days	0.25	0.14	1.1	0.7	1.4	10.6	18	15	15	25
Mix aged 7 days	0.10	0.24	0	1.4	2.1	13.4	29	29	48	48
Average loss on original mix and mix aged 7 days	0.13	0.18	0	0.9	1.2	7.4	15	16	24.5	25.5

[1] Residue from thin-film oven test ⅛-in. film, 5 hours at 325° F.

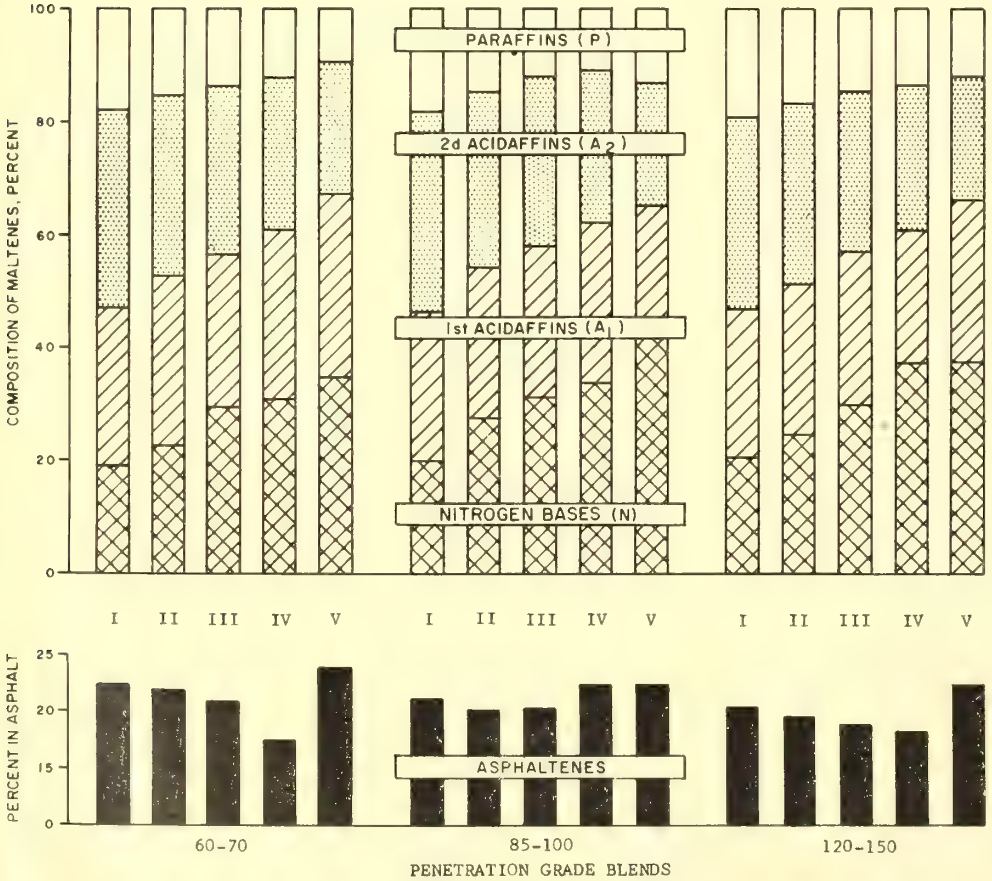

Figure 4.—Composition of asphalt blends shown by percent of asphaltenes in the blend and by composition of maltenes.

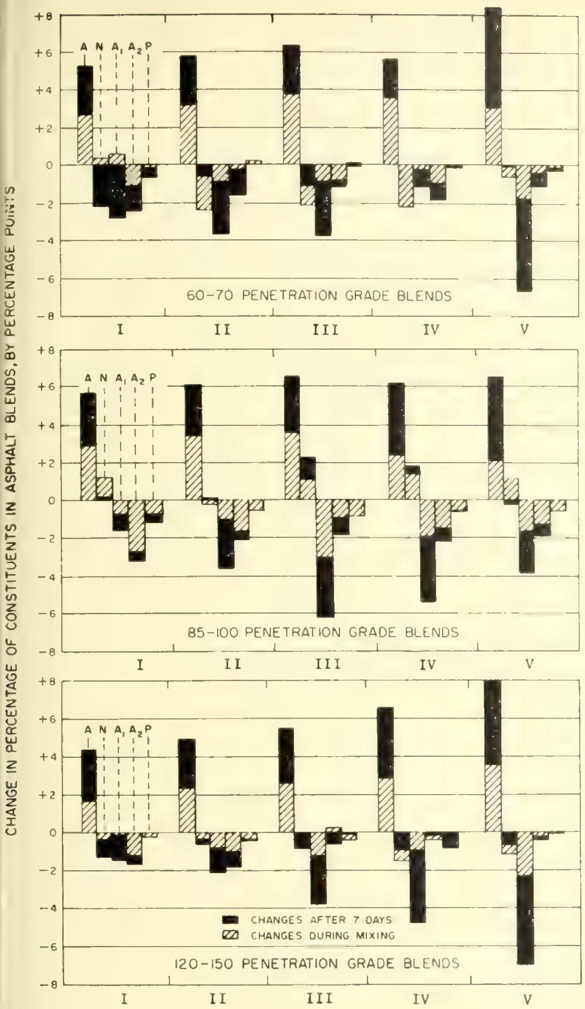

Figure 5.—Changes in composition of asphalt blends during mixing and after 7 days of aging at 140° F.

amount of asphaltenes or the average molecular weights of either the maltenes constituents or asphaltenes have not been evaluated by these tests as these properties did not vary substantially in the series of specimens tested. Factors such as those mentioned most likely have an important effect on the overall behavior of an asphalt as a binder in a pavement.

The points represented by open circles on figure 8 are for the asphalts graded by viscosity at 140° F. These data are for the same asphalts as in figure 3, which showed no relationship between abrasion loss and value of the compositional ratio $(N + A_1)/(P + A_2)$. Figure 8 data show that the lack of correlation was a result of widely different viscosities at 77° F. When the abrasion loss was measured at temperatures at which the asphalts had the same viscosity and was plotted against the composition parameter, good correlation was obtained.

Analyses of Data

As part of the contract under which the research discussed here was performed, some

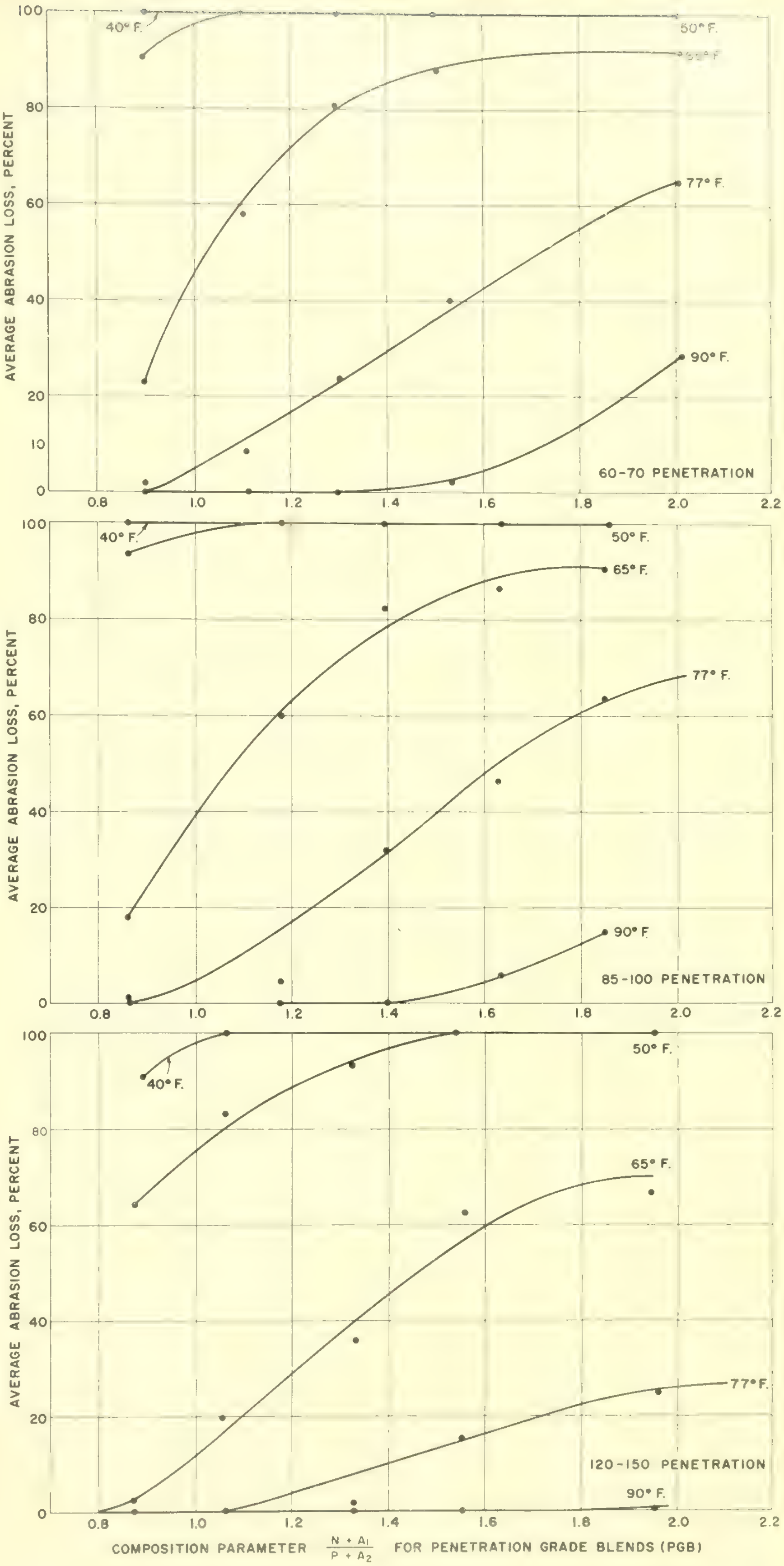

Figure 6.—Relation of average abrasion loss at different temperatures to composition parameter for blends of indicated penetration grades of asphalts.

Table 8.—Components and chemical properties of asphalt blends

	Group I				Group II				Group III				Group IV				Group V			
	Blend	Thin-film oven test residue	Asphalt recovered from Original mix	Asphalt recovered from Mix aged 7 days	Blend	Thin-film oven test residue	Asphalt recovered from— Original mix	Asphalt recovered from— Mix aged 7 days	Blend	Thin-film oven test residue	Asphalt recovered from— Original mix	Asphalt recovered from— Mix aged 7 days	Blend	Thin-film oven test residue	Asphalt recovered from— Original mix	Asphalt recovered from— Mix aged 7 days	Blend	Thin-film oven test residue	Asphalt recovered from— Original mix	Asphalt recovered from— Mix aged 7 days
60-70 PENETRATION GRADE BLENDS																				
Components:																				
A.........pct. by wt	24.8	27.1	27.5	30.1	24.2	26.9	27.4	30.0	21.8	24.0	25.5	28.1	14.9	17.6	18.4	20.5	27.9	31.1	30.9	36.2
N.........do	14.4	13.1	12.2	14.8	17.0	15.8	14.7	16.5	23.0	22.0	20.9	21.8	26.5	25.1	24.3	24.2	24.7	24.0	24.0	24.6
A_1.........do	21.1	20.9	21.8	18.4	22.8	21.4	22.0	19.1	21.1	20.0	20.2	17.3	24.9	24.0	24.6	23.7	23.5	21.4	21.7	16.7
A_2.........do	26.1	25.4	25.0	23.7	24.2	24.4	24.0	22.6	23.2	23.2	22.4	22.0	23.3	23.1	22.3	21.4	17.2	17.1	16.8	16.1
P.........do	13.6	13.5	13.5	13.0	11.8	11.5	11.9	11.8	10.9	10.8	11.0	10.8	10.4	10.2	10.4	10.2	6.7	6.4	6.6	6.4
$\frac{N+A_1}{P+A_2}$	0.89	0.87	0.88	0.90	1.11	1.04	1.02	1.03	1.29	1.24	1.23	1.19	1.53	1.47	1.50	1.52	2.02	1.93	1.95	1.84
Paraffin fraction:																				
Wax indication [1]	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT
Index of refraction	1.4812	1.4817	1.4813	1.4808	1.4815	1.4821	1.4826	1.4824	1.4826	1.4829	1.4833	1.4832	1.4849	1.4844	1.4843	1.4844	1.4812	1.4810	1.4812	1.4809
85-100 PENETRATION GRADE BLENDS																				
Components:																				
A.........pct. by wt	22.6	25.3	25.5	28.3	20.4	23.7	23.8	26.5	20.6	23.8	24.2	27.1	24.7	27.2	27.1	30.9	24.7	26.9	26.8	31.2
N.........do	15.4	16.1	16.6	15.6	22.1	21.7	21.9	22.2	24.9	26.2	26.0	27.2	25.5	27.4	26.9	27.3	31.4	32.6	32.6	31.2
A_1.........do	20.5	20.3	19.8	18.9	21.0	20.0	20.0	17.4	21.4	18.9	18.4	15.2	21.3	18.6	19.4	16.0	17.6	15.9	16.0	13.7
A_2.........do	27.5	24.7	24.8	24.3	24.8	22.9	23.2	22.8	23.5	21.9	22.6	21.7	20.2	19.0	18.8	18.1	16.2	15.0	15.0	14.3
P.........do	14.0	13.6	13.3	12.9	11.7	11.7	11.1	11.1	9.6	9.2	8.8	8.8	8.3	7.8	7.8	7.7	10.1	9.6	9.6	9.6
$\frac{N+A_1}{P+A_2}$	0.87	0.95	0.96	0.93	1.18	1.21	1.22	1.17	1.40	1.45	1.41	1.39	1.64	1.72	1.74	1.68	1.86	1.97	1.98	1.88
Paraffin fraction:																				
Wax indication	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT
Index of refraction	1.4814	1.4815	1.4814	1.4822	1.4825	1.4830	1.4833	1.4833	1.4823	1.4817	1.4821	1.4822	1.4810	1.4816	1.4810	1.4815	1.4823	1.4830	1.4829	1.4828
120-150 PENETRATION GRADE BLENDS																				
Components:																				
A.........pct. by wt	21.2	23.0	22.9	25.6	19.3	21.2	21.7	24.2	17.9	19.7	20.2	23.4	16.9	19.4	19.8	23.5	24.8	27.5	28.4	32.8
N.........do	16.1	15.9	15.8	14.9	20.0	19.0	19.7	19.4	24.8	24.0	23.8	23.9	31.3	30.4	29.8	30.4	28.4	27.3	27.2	27.8
A_1.........do	20.8	20.7	20.7	19.4	21.6	21.0	20.8	19.5	22.1	22.1	20.9	18.2	19.2	18.1	18.3	14.4	21.4	19.8	19.1	14.4
A_2.........do	26.8	25.1	25.7	25.2	25.7	24.9	24.7	23.9	23.3	22.4	23.6	22.7	21.4	21.0	21.0	21.3	16.3	16.3	16.2	16.0
P.........do	15.1	15.3	14.9	14.9	13.4	13.9	13.1	13.0	11.9	11.8	11.5	11.8	11.2	11.1	11.1	10.4	9.1	9.1	9.1	9.0
$\frac{N+A_1}{P+A_2}$	0.88	0.91	0.90	0.86	1.06	1.03	1.07	1.05	1.33	1.35	1.27	1.22	1.55	1.51	1.50	1.41	1.96	1.85	1.83	1.69
Paraffin fraction:																				
Wax indication [1]	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT	+RT
Index of refraction	1.4822	1.4828	1.4820	1.4822	1.4816	1.4825	1.4816	1.4816	1.4836	1.4831	1.4840	1.4834	1.4828	1.4830	1.4824	1.4827	1.4804	1.4812	1.4802	1.4801

[1] +RT represents the presence of wax at room temperature.

Table 9.—Abrasion characteristics shown by pellet abrasion tests on asphalt blends [1]

Pellet abrasion test temperature, ° F.	Group I, percent loss in test				Group II, percent loss in test				Group III, percent loss in test				Group IV, percent loss in test				Group V, percent loss in test			
	Original mix	Mix aged 3 days	Mix aged 7 days	Average, original mix and mix aged 7 days	Original mix	Mix aged 3 days	Mix aged 7 days	Average, original mix and mix aged 7 days	Original mix	Mix aged 3 days	Mix aged 7 days	Average, original mix and mix aged 7 days	Original mix	Mix aged 3 days	Mix aged 7 days	Average, original mix and mix aged 7 days	Original mix	Mix aged 3 days	Mix aged 7 days	Average, original mix and mix aged 7 days
60-70 PENETRATION GRADE BLENDS																				
40	100(278)	100(264)	100(249)	100	100	100(198)	100(105)	100	100	100(150)	100(103)	100	100	100(118)	100(110)	100	100	100(72)	100(93)	100
50	79	100(416)	100(303)	90+	100(323)	100(223)	100(195)	100	100(191)	100(143)	100(165)	100	100(154)	100(165)	100(126)	100	100(280)	100(158)	100(88)	100
65	9	13	36	22.5	34	67	80	57	61	87	100(441)	81+	73	100(440)	100(418)	87+	81	100(279)	100(211)	91+
77	0.35	0.5	2.4	1.9	1.1	1.1	13.0	7.0	3.6	9	45	24	17	49	62	40	33	74	96	65
90	0.12	0.0	0.05	0.1	0.12	0.12	0.20	0.2	0.05	0.18	0.80	0.4	0.45	0.7	2.7	1.6	1.1	12	53	27
85-100 PENETRATION GRADE BLENDS																				
40	100(347)	100(260)	100(184)	100	100(282)	100(130)	100(112)	100	100(174)	100(80)	100(63)	100	100(170)	100(91)	100(70)	100	100(125)	100(80)	100(43)	100
50	87	93	100(410)	94+	99	100(240)	100(200)	100	100(250)	100(171)	100(100)	100	100(246)	100(142)	100(115)	100	100(246)	100(134)	100(94)	100
65	7	24	29	18	33	65	87	60	64	96	100(395)	82	72	98	100(310)	86+	79	100(283)	100(94)	90+
77	0.3	0.6	0.9	0.6	0.7	2.1	6.0	3.4	10	27	53	31	22	49	69	46	29	77	96	63
90	0	0.1	0.3	0.1	0	0.2	0.1	0.1	0	0.7	0.2	0.1	0.3	1.6	11	5.6	0.4	8.7	29	15
120-150 PENETRATION GRADE BLENDS																				
40	81	100(338)	100(302)	91+	100(399)	100(235)	100(216)	100	100(375)	100(224)	100(173)	100	100(326)	100(208)	100(140)	100	96	100(127)	100(96)	98+
50	43	66	86	65	66	99	100(440)	83	86	100(253)	100(318)	93+	100(418)	100(227)	100(203)	100	100(485)	100(315)	100(215)	100
65	0.3	2.2	3.4	1.8	4.6	22	34	19	12	39	61	36	35	69	90	62	34	76	100(403)	67
77	0.15	0.25	0.1	0.12	0.1	1.1	0	0.1	0.2	1.4	2.1	1.2	0.9	18	29	15	0.9	15	48	24
90	0.02	0.02	0.10	0.06	0	0.02	0.02	0	0.02	0.10	0.10	0.06	0.05	0.5	0.4	0.22	0.15	0.5	1.3	0.72

[1] Figures in parentheses are the number of revolutions to 100 percent loss.

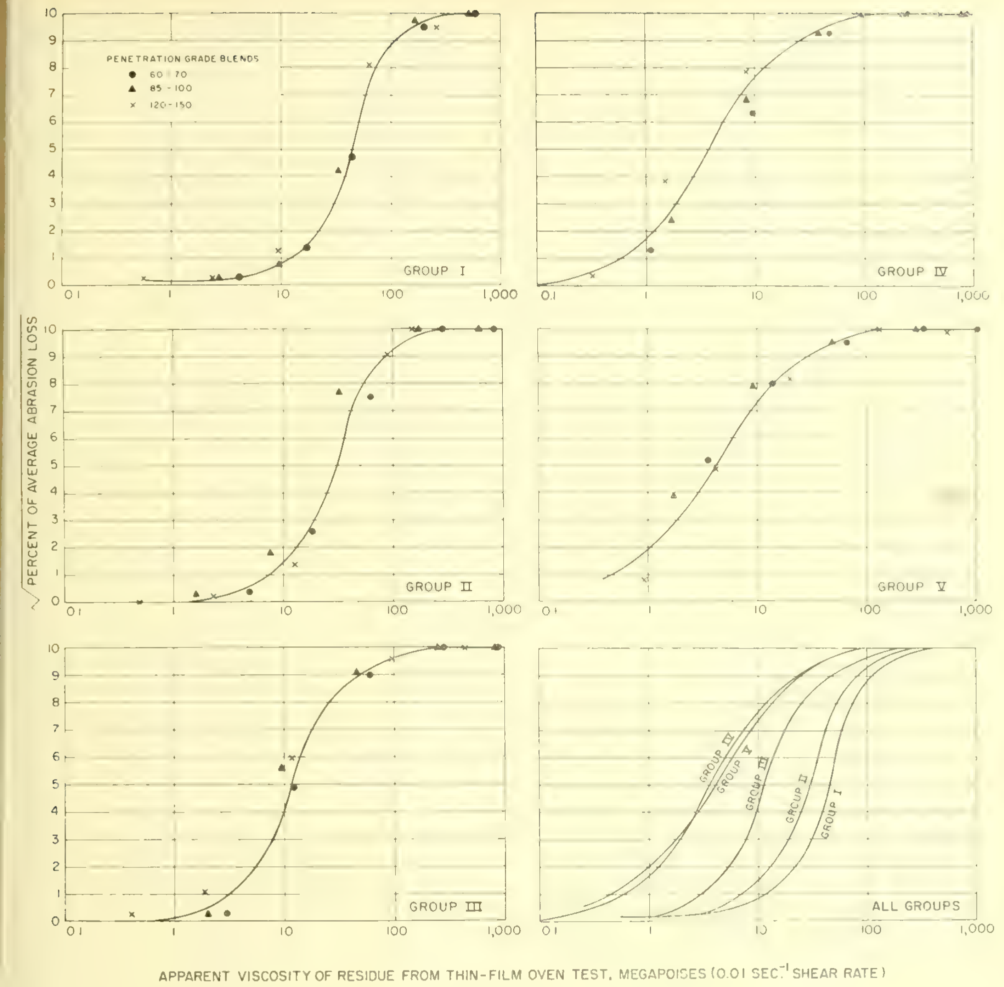

Figure 7.—Relation of abrasion loss to viscosity of thin-film oven test residue at abrasion test temperature.

analyses of data were conducted that have not been reported in detail. One of these was a computer analysis of the data to determine whether a more significant relation between the abrasion resistance and composition could be obtained mathematically, rather than experimentally. Relations with single constituents or ratios of single constituents were explored. Results showed that the ratio N/P indicated the best correlation for single constituents and corresponded closely to results obtained by use of $(N+A_1)/(P+A_2)$ with the latter ratio being slightly superior. The effect of changing coefficients for $N+A_1$ and $P+A_2$ was also explored, but no improvement over the 1 to 1 ratio was obtained. An attempt to find mathematically the best fit for the data by calculating a power index gave the form:

$$\text{Power index}=K\frac{(N/P+8)^{4.82}(A_1/N-0.7)^{0.14}}{(A_2/P)^{0.24}}$$

Obviously such an expression has no practical significance nor is it theoretically founded. In any event, plotting this parameter and others obtained by computer analyses showed no advantage over the parameter $(N+A_1)/(P+A_2)$ arrived at from chemical considerations. The exploratory computer analysis conducted was concerned only with relations of the different fractions in the maltenes to abrasion resistance measured by the pellet abrasion test. Consideration was not given, in the study reported here, to the relation of chemical constitution to other characteristics of the asphalt or the effects of asphaltenes.

Another analysis of the data involved the computation of a general linear equation to calculate abrasion loss as a function of viscosity and composition. The data measured on the composite blends was the basis of the computations. Calculations were made using the portion of the *S*-curve for square root of average abrasion loss related to viscosity in poises at 0.05/sec. shear rate, which approximated a straight line—the portion between 1 and 99 percent. Using the least squares method for obtaining the best fit, separate curves for each group of asphalts were calculated as follows:

Group I:
$$\sqrt{\text{abrasion loss}} \quad 6.74 \log \eta -42.6$$

Group II:
$$\sqrt{\text{average abrasion loss}}= 5.95 \log \eta -34.8$$

Group III:
$$\sqrt{\text{average abrasion loss}}=4.68 \log \eta -24.6$$

Group IV:
$$\sqrt{\text{average abrasion loss}}=4.93 \log \eta -24.9$$

Group V:
$$\sqrt{\text{average abrasion loss}}=3.67 \log \eta -16.1$$

It is apparent from these equations that the slope of the line and its relative location is a function of the value of the ratio $(N+A_1)/(P+A_2)$, as this is the difference between the groups. When the slopes and constants were plotted against the value of the ratio for each blend, a general relation was obtained by which the five equations could be reduced to a single equation.

$$\sqrt{\text{Average abrasion loss}} =\log \eta \left(9-2.75\frac{N+A_1}{P+A_2}\right)+24\left(\frac{N+A_1}{P+A_2}\right)-61.5$$

This equation provides a means for predicting the abrasion resistance when the composition and viscosity of an asphalt are known. Comparisons of measured data with calculated data showed that for the blends most results agreed to within ±15 percent, and for the viscosity graded asphalts result agreements were within the band of ±20 percent. Figures 9 and 10, respectively, show these two relationships. Although calculated results greater than 100 percent or less than zero have no true significance, these results are shown in figures 9 and 10 for comparison. This degree of agreement between calculated and measured results suggests that an alternate procedure for measuring abrasion resistance could be devised that would permit better overall evaluation. Some preliminary studies were conducted using the weight loss per revolution for such an evaluation. This approach eliminates the zero and 100 percent loss figures and provides characteristic values for all asphalts.

Figure 11 data are an example of the improvement in test results available by use of this refinement in the abrasion test. The curves show percent average abrasion loss per revolution against the parameter $(N+A_1)/(P+A_2)$ for the 120–150 penetration grade composite blends. The individual curves are for the different temperatures at which the tests were run. The improvement obtained by this approach to measuring abrasion resistance is strikingly brought out by the fact that all data above the line of 0.2 percent loss per revolution were previously reported as 100 percent loss (fig. 6). The same general pattern was shown by data from other penetration grades.

Chemical Composition Related to Physical Characteristics

The relationship between the ratio $(N+A_1)/(P+A_2)$ and the durability of an asphalt as a cementing agent was originally postulated on the basis of the logical assumption that durability must be related to a parameter expressive of the ratio of reactive to nonreactive components. The product is less durable and more susceptible to embrittlement as more of the reactive components $(N+A_1)$ are present in an asphalt. It was neither postulated nor assumed that the parameter $(N+A_1)/(P+A_2)$, should be or could be indicative of any property of asphalts except one that logically can be related to chemical reactivity. No attempt was made, therefore, in the study reported he to correlate this parameter with any property other than embrittlement.

It can, however, be expected that parameters also can be developed to relate other properties, such as viscosity characteristics, to composition of the asphalt. A parameter for viscosity characteristics logically would have to be based on viscosity characteristics of the individual components. It can be postulated that in a parameter relating viscosity characteristics to composition the components, A, P, and N and their molecular weights should have significant effects on the relation. The data in this article provide some of the measured values that can be used in such further research, but they are not sufficient for determining the specific relations that might exist.

The data presented for the viscosity graded asphalts provide information on the chemical reactivity of the components of these asphalts and are available for comparison with other data now being developed by other researchers. These other data include chromatographic separations, molecular weights, and interactions with aggregates. Hopefully, the additional information will permit a sorting out of all important relations so that a clear understanding of the role of chemical composition on the performance of asphalt eventually can be attained.

Summary

A significant effect of asphalt composition was determined, by the precipitation method of fractional analysis for asphalts, from data collected in the study discussed here. The viscosity of the asphalt significantly affected the results of the pellet abrasion test but comparisons made on the basis of the same asphalt viscosity also showed that abrasion resistance decreased as the value of the parameter $(N+A_1)/(P+A_2)$ increased from a minimum of about 0.5 to the maximum measured in this study, 2.24. In other work by Rostler and White they have reported that for synthetic asphalts abrasion resistance increases rapidly as the ratio falls below 0.4 because the excess of saturated or nearly saturated components tends to destroy the cohesive forces within the asphalt. Similarly, very poor abrasion resistance is the result of an excess of highly reactive components that degrade rapidly.

Data are not now sufficient to establish exact limits for the parameter $(N+A_1)/(P+A_2)$, that will provide for acceptable field performance of an asphalt as a highway binder. However, evidence indicates that a ratio within a range of 0.8 to 1.5 produces materials of good original and retained cementing quality. The spread of results for abrasion resistance for asphalts having equal viscosities and equal values of the parameter, as well as a lack of general correlation of parameter value with other physical test characteristics of the asphalts, emphasizes the role of factors not measured by these tests—such factors as the quantitative amount of asphaltenes and the molecular weights of all constituents.

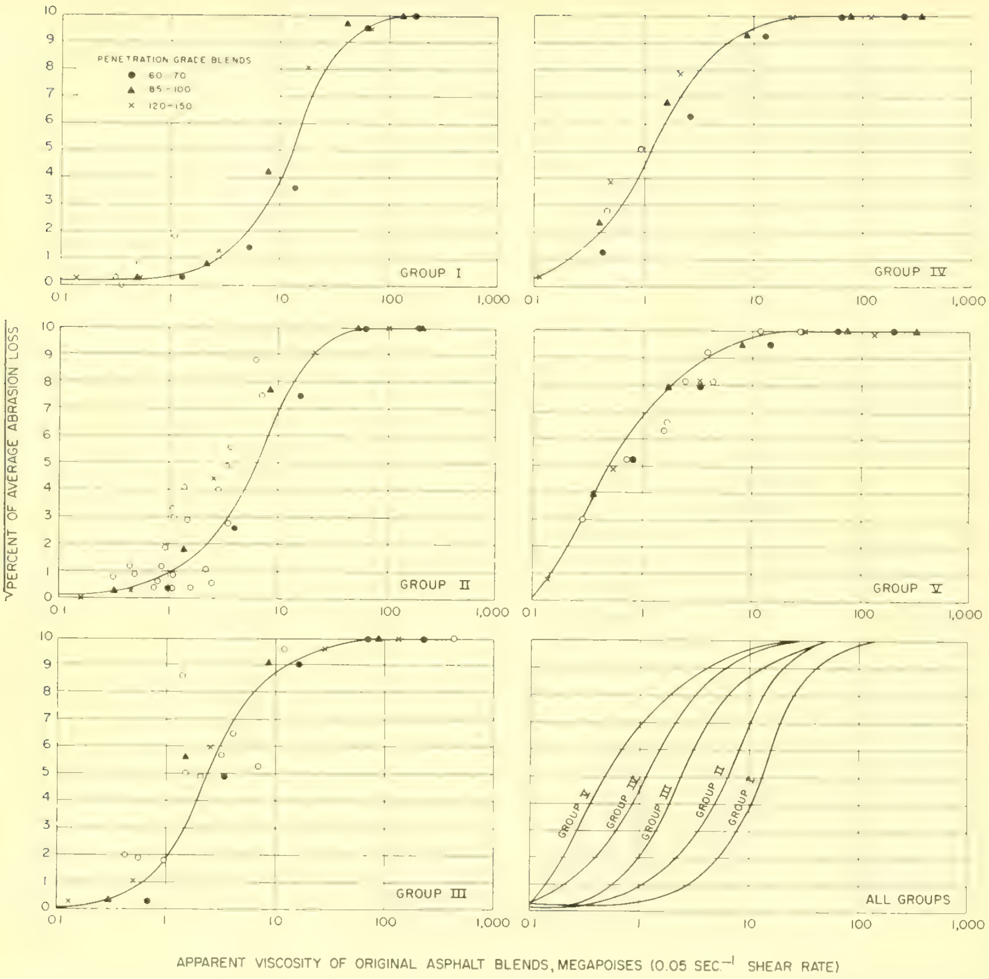

Figure 8.—Relation of abrasion loss to original asphalt viscosity at abrasion test temperatures.

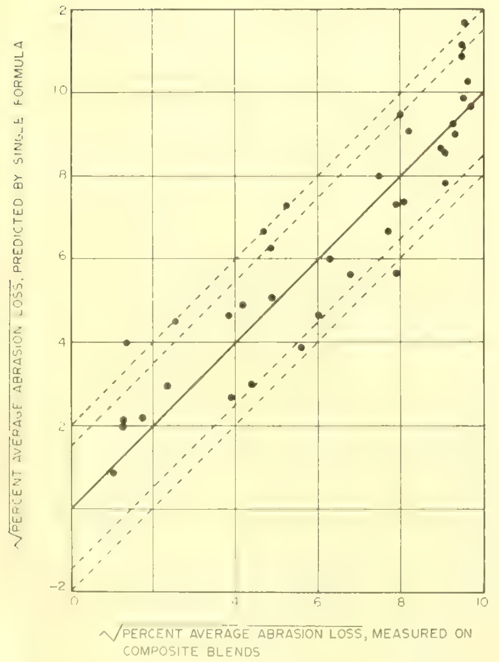

Figure 9.—Relation of abrasion loss predicted from single formula to measured abrasion loss of blends of penetration grade asphalts.

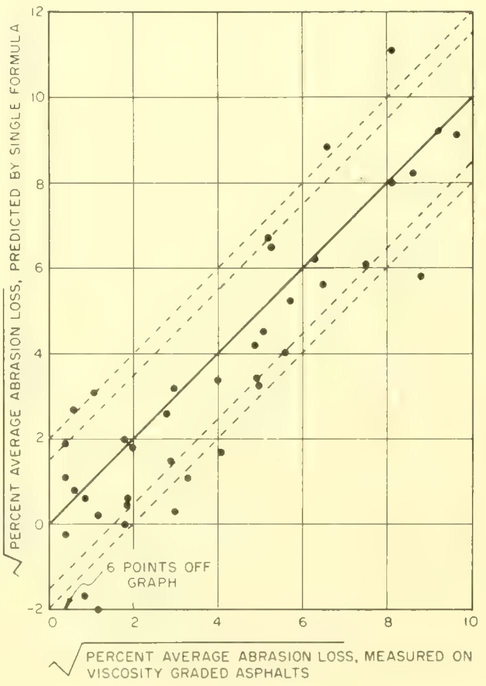

Figure 10.—Relation of abrasion loss predicted from single formula to measured abrasion loss of viscosity graded asphalts.

An equation, taking into account, both consistency and chemical composition of the asphalt has been developed. This equation represents a first approach to a generally applicable mathematical expression for determining the inherent susceptibility of an asphalt to embrittlement. The fact that data obtained on asphalts of widely different origin, consistency, and composition could be fitted into a reasonably narrow band illustrated the dependence of durability on both composition and viscosity.

Physical and chemical data believed to be characteristic of present day asphalts have been measured on a large collection of asphalt samples. These data provide an extensive accumulation of figures that can be further analyzed and compared with findings of other workers in the field.

ASPHALT IDENTIFICATION

For the convenience of researchers who may wish to compare data from this study with data from other studies, a cross-reference for the different identification numbers of the viscosity graded asphalts is shown in the following tabulation. The original Bureau of Public Roads numbers and the numbers assigned by The Asphalt Institute are shown.

Asphalt code number	*BPR laboratory number*	*The Asphalt Institute number*
A-5	B-2908	3
B-5	B-2920	31
C-5	B-2958	4
D-5	B-2962	33
E-5	B-2974	1
F-5	B-3008	2
G-5	B-3012	13
H-5	B-3028	35
I-5	B-3037	37
J-5	B-3050	22
K-5	B-3054	23
L-5	B-3058	28
M-5	B-3108	39
N-5	B-3578	46
O-5	B-3601	49
A-10	B-2909	7
B-10	B-2921	14
C-10	B-2959	8
D-10	B-2963	21
E-10	B-2975	5
F-10	B-3009	6
G-10	B-3013	20
H-10	B-3029	19
I-10	B-3036	15
J-10	B-3051	16
K-10	B-3055	17
L-10	B-3059	18
M-10	B-3109	40
N-10	B-3579	47
O-10	B-3602	50
A-20	B-2910	11
B-20	B-2922	32
C-20	B-2960	12
D-20	B-2964	34
F-20	B-3010	10
G-20	B-3014	38
H-20	B-3030	9
I-20	B-3035	36
J-20	B-3052	24
K-20	B-3056	25
L-20	B-3060	29
M-20	B-3110	41
N-20	B-3580	48
O-20	B-3603	51
P-20	B-3039	43
J-40	B-3053	26
K-40	B-3057	27
L-40	B-3061	30
O-40	B-3604	52
P-40	B-3040	44

Table 10.—Viscosity of asphalt blends at different temperatures and shear rates

Temperature, ° F.	Shear rate sec.⁻¹	Viscosity, megapoises									
		Group I		Group II		Group III		Group IV		Group V	
		Blend	Thin-film residue [1]	Blend	Thin-film residue [1]	Blend	Thin-film residue [1]	Blend	Thin-film residue [1]	Blend	Thin-film residue [1]
60-70 PENETRATION GRADE ASPHALT BLENDS											
90	0.1	1.23	2.57	0.98	3.48	0.63	2.45	0.43	1.10	0.70	2.55
	0.05	1.32	2.97	1.02	3.85	0.68	2.62	0.42	1.10	0.78	2.82
	0.01	1.60	4.20	1.09	4.80	0.69	3.05	0.40	1.10	0.97	3.50
	0.001	2.08	6.85	1.22	6.60	0.72	3.78	0.38	1.10	1.33	4.85
77	0.1	5.15	9.24	3.90	8.50	3.30	8.95	2.66	9.2	3.15	8.60
	0.05	5.35	11.1	4.10	10.6	3.45	10.0	2.67	9.3	3.45	9.90
	0.01	5.70	16.7	4.70	17.9	3.88	12.5	2.69	9.6	4.25	13.5
	0.001	6.35	30.0	5.70	37.5	4.55	17.5	2.70	10.0	5.70	21.1
65	0.1	12.1	18.3	14.3	31.3	14.8	36.2	12.0	28.6	13.7	50.0
	0.05	14.3	24.0	16.5	38.5	16.5	62.0	12.8	33.5	15.0	55.0
	0.01	21.4	44.5	23.8	42.0	20.5	60.0	15.0	48.0	18.5	67.5
	0.001	37.3	108	39.5	123	28.1	100	18.8	80.5	25.0	91.0
39.2	0.1	122	158	138	255	169	279	174	300	141	353
	0.05	172	232	195	370	227	390	237	420	198	495
	0.01	380	580	430	830	450	855	485	910	430	1,080
	0.001	1,200	2,150	1,360	2,725	1,200	2,650	1,340	2,780	1,320	3,275
85-100 PENETRATION GRADE ASPHALT BLENDS											
90	0.1	0.48	1.87	0.30	1.20	0.30	1.55	0.36	1.37	0.33	1.52
	0.05	0.50	2.10	0.32	1.32	0.30	1.67	0.39	1.44	0.36	1.57
	0.01	0.55	2.70	0.36	1.64	0.30	1.98	0.47	1.70	0.43	1.70
	0.001	0.63	3.90	0.44	2.23	0.31	2.53	0.62	1.90	0.55	1.90
77	0.1	2.02	5.35	1.38	5.10	1.47	7.20	1.57	6.05	1.64	7.20
	0.05	2.20	6.35	1.38	5.75	1.47	7.80	1.65	6.65	1.71	7.80
	0.01	2.65	9.55	1.38	7.65	1.47	9.35	1.83	8.30	1.93	9.45
	0.001	3.50	17.2	1.38	11.5	1.47	12.1	2.12	11.3	2.29	12.3
65	0.1	7.30	13.8	8.00	20.0	8.35	31.3	8.05	29.6	7.60	36.2
	0.05	7.80	17.8	8.35	23.2	8.70	35.2	8.65	31.7	8.00	40.0
	0.01	9.15	32.0	9.05	32.5	9.55	46.0	10.2	37.5	8.95	50.5
	0.001	11.50	73.5	10.3	52.5	11.0	68.5	12.8	47.5	10.5	70.5
39.2	0.1	96.0	153	163	202	375	310	292	315	259	435
	0.05	135	222	205	280	435	420	345	420	318	570
	0.01	280	525	350	600	615	840	505	825	510	1,070
	0.001	800	1,800	745	1,800	1,010	2,280	885	2,160	1,020	2,620
120-150 PENETRATION GRADE ASPHALT BLENDS											
90	0.1	0.13	0.54	0.15	0.46	0.12	0.37	0.11	0.28	0.13	0.82
	0.05	0.15	0.54	0.16	0.47	0.13	0.38	0.11	0.29	0.14	0.85
	0.01	0.18	0.56	0.18	0.49	0.17	0.41	0.11	0.32	0.16	0.94
	0.001	0.24	0.59	0.23	0.53	0.24	0.43	0.11	0.36	0.21	1.09
77	0.1	0.53	1.86	0.44	1.90	0.47	1.90	0.49	1.44	0.53	3.10
	0.05	0.55	2.00	0.46	2.00	0.50	1.90	0.50	1.47	0.56	3.40
	0.01	0.59	2.41	0.50	2.26	0.57	1.90	0.52	1.53	0.63	4.20
	0.001	0.65	3.12	0.58	2.70	0.69	1.90	0.56	1.62	0.76	5.70
65	0.1	2.65	5.08	2.60	7.55	2.46	9.48	2.14	8.40	3.38	12.3
	0.05	2.75	6.15	2.62	8.95	2.57	10.2	2.16	8.40	3.38	14.2
	0.01	3.02	9.55	2.70	13.1	2.88	12.1	2.22	8.40	3.62	20.3
	0.001	3.45	18.1	2.82	22.5	3.37	15.3	2.30	8.40	4.02	33.5
39.2	0.1	56.7	94	88.5	152	113	180	108	310	118	240
	0.05	69.5	128	105	200	132	236	120	362	135	308
	0.01	112	260	153	380	190	435	153	510	182	555
	0.001	216	715	265	960	317	1,040	215	850	276	1,300

[1] Residue from thin-film oven test, ⅛-in. film, 5 hours at 325° F.

TEST PROCEDURES

To maintain the continuity of the work, the test procedures used in the research reported here were basically the same used in the previous investigations. The refinements and improvements employed are described in the following paragraphs.

Chemical

The chemical composition of the asphalts was determined by the precipitation method described in detail in reference 3. The only change in the procedure was that the time between treatment of the sample with 97- to 98-percent H_2SO_4 and the following step of decanting and [illegible] the solution was limited to a maximum of 3 hours. The improvement brought about by this refinement has been explained previously (6).

Preparation of Ottawa sand and asphalt mixtures

The procedure described in reference (7) was used in the preparation of Ottawa sand and asphalt mixture, except that the batch size was larger, namely, 300 grams of sand and 6 grams of asphalt.

Aging sand-asphalt mixtures

For aging the sand-asphalt mixtures, a larger cabinet and slightly different pans to

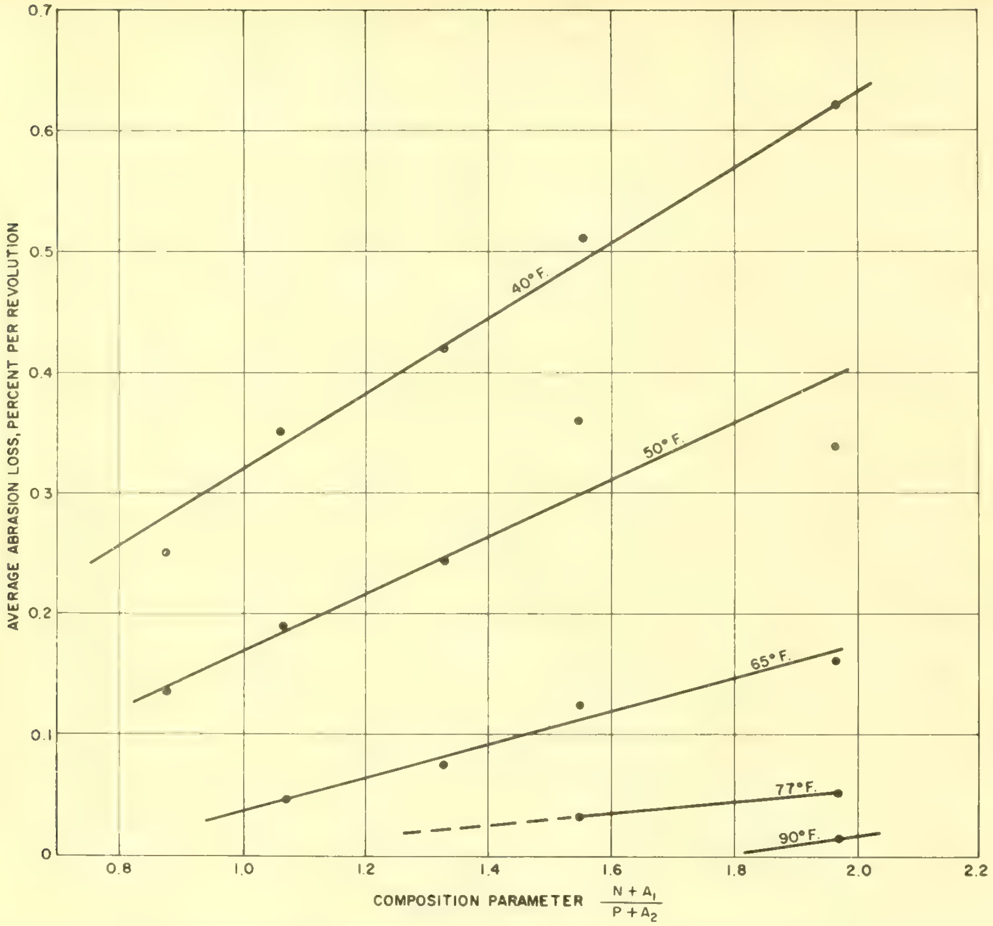

Figure 11.—Relation of average abrasion loss in percent per revolution of test device to composition parameter at different test temperatures for 120–150 penetration grade blends.

holding the specimen were used than described in reference (7). A photograph of the cabinet is shown in figure 12. The cabinet was designed to give aging effects identical to those obtained in the cabinet used previously. In calibrating the cabinet, a number of parallel aging tests were performed and the openings regulating air circulation were adjusted until the conditions represented identical aging environments in the test cabinets.

The apparatus and procedure outlined for use by laboratory technicians is as follows:

Aging cabinet. Steel, 28 by 28 by 36 inches high, containing a 25-inch diameter turntable rotating 6 r.p.m., and four 250-watt infrared reflector lamps thermostatically controlled to maintain sand-asphalt mixture at a temperature of 140 ± 2° F.

Sample pans. Aluminum, 97 mm. in diameter and 15 mm. deep.

Thermometers. Dial, 1¾-inch diameter having a 5-inch stem and a temperature range of 0° to 180° F., with the stem painted dull black. Two or three aging pans should be fitted with these thermometers. The stem of the thermometer should be inserted through a ⅛-inch diameter hole drilled in the side, ¼ inch above the bottom of the pan, with the end of the stem supported in a depression made by denting the side of the pan diametrically opposite the hole. Epoxy cement should be used for fastening the thermometers in place.

Calibration

For proper calibration, the aging cabinet must be operated where the room temperature can be maintained at 77 ± 4° F.—a location away from windows, free from drafts and from direct sunlight.

To complete the calibration procedure, load the turntable with 15 aging sample pans each containing 60 grams of Ottawa sand-asphalt mix. A full load of samples is necessary to ensure correct equilibrium between radiant heat absorbed and heat lost from all causes. Two or three of the pans should have dial thermometers. Hang the thermometer dials over the edge of the turntable in a proper position to avoid striking the walls as the table turns. Adjust the variable transformer so that the three lamps controlled by it will maintain a temperature of about 130° F. in the pans; then adjust the thermostat so that the fourth lamp, operating intermittently, maintains the temperature at 140 ± 2° F.

Procedure

Procedure for completing the testing is to: Preheat the aging cabinet at least 1 hour. Weigh 60 ± 0.2 grams Ottawa sand-asphalt mixture into an identified aging sample pan, trowelling the surface reasonably smooth with a warm spatula. Place the sample pans on the turntable, between 7½ and 12 inches from the center. If less than 15 specimens are tested, ballast the turntable with extra pans of sand-asphalt mix to be kept on hand for the purpose. If some specimens are removed at intervals during the aging period, replace them with extra pans of mix so that there are always 15 pans of mix on the table. Do not stir or remove any portion of a specimen during the aging period. At the end of the aging period, remove the pan, mix the contents gently with a spatula for 30 seconds and transfer them to an airtight container.

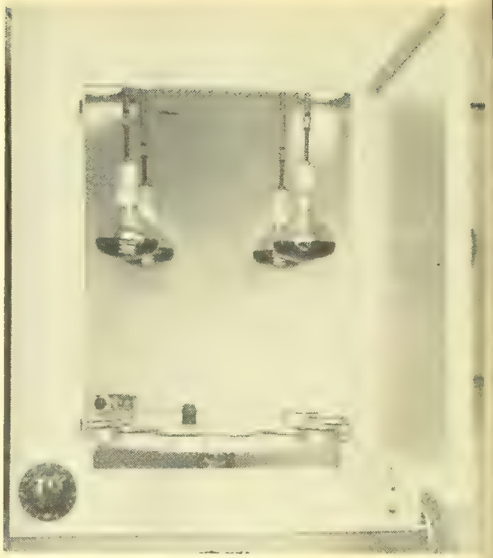

Figure 12.—Infrared aging oven.

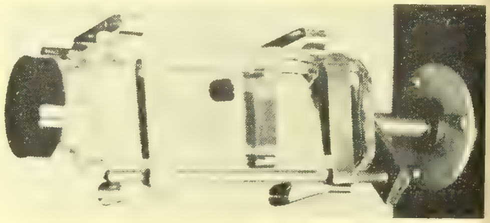

Figure 13.—Spring-clip bottle holder.

Pellet Abrasion Test

The apparatus used for the pellet abrasion test for Ottawa sand-asphalt mixtures at different temperatures, was as follows:

Mold, from pellet press.

Carver laboratory press.

Force gage, proving ring, and other attachments, used on Carver press permitting accurate gage reading in the range used, including that produced by a force of 200 pounds on the pellet ram.

Bottle, wide-mouth French square, 16-ounce capacity, with screw cap, see figure 13.

Universal timer, Gralab, laboratory model.

Weighing dishes, disposable aluminum.

Analytical balance.

Thermometer, range dependent on test temperature.

Temperature-controlled cabinet, 8- by 8-inch multipane window in front, access port in side, circulating fan, and 75-watt incandescent light, figure 14.

Bottle rotating device, a spring-clip bottle holder mounted on shaft extending through access port to externally mounted Boston gear ratiomotor, VMB 5820-S, 87.5 r.p.m., figure 13.

Calibration

For proper calibration, determine the time required for the bottle rotating device to rotate 500±10 revolutions. One hour before testing, set the temperature control to maintain the desired temperature in the cabinet ±1° F. Use a 75-watt incandescent light bulb to maintain close temperature control and to provide sufficient heat for operating above room temperature up to 90° F. If higher temperatures are required, an additional heat source is required.

Procedure

Procedure for this pellet abrasion test is to weigh out two, 2±0.1-gram portions of specified Ottawa sand-asphalt mixture, or measure portions using 7/16-inch I.D. arch-punch fitted with sliding piston and adjustable stop calibrated to deliver a 2±0.1-gram portion of mix, figure 15. Mold each portion into a pellet, maintaining for 1 minute a force of 200 pounds on the pellet ram, equivalent to 1,000 p.s.i. on the ½-inch diameter pellet. Allow pellets to rest at least one-half hour before abrading. Pellets not abraded the same day should be discarded or broken up and returned to sealed sample cans for later molding.

Weigh each pellet, in an aluminum dish, to the nearest 0.001 gram. Carefully place one pellet in a square bottle that is in a horizontal position. Use long-handled spoon (iced-tea spoon) for inserting and removing pellets. Close the bottle and allow to remain at test temperature for one-half hour (omit waiting period for tests made at room temperature). Place the bottle in the holding device; check temperature and allow it to stabilize again if necessary. Set the timer to rotate the bottle for 500 revolutions; observe through the viewing window to be sure that the pellet is free to tumble, that is, not stuck to bottle. At the end of the tumbling period, carefully remove the largest remaining piece of the pellet, place it in the aluminum dish and weigh it to the nearest 0.001 gram. If the pellet disintegrates completely before the end of the test, stop the rotation and record the elapsed time.

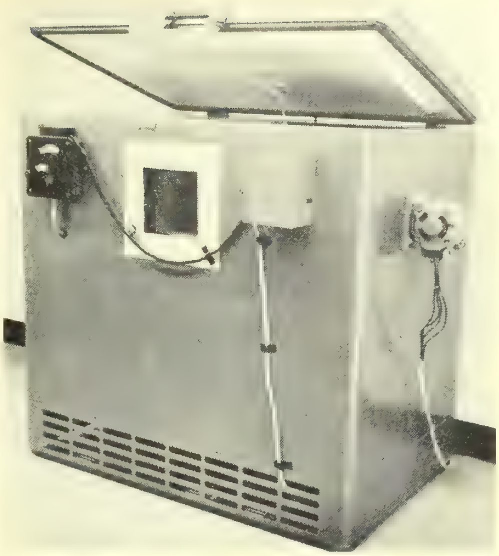

Figure 14.—Temperature-controlled (cold) cabinet.

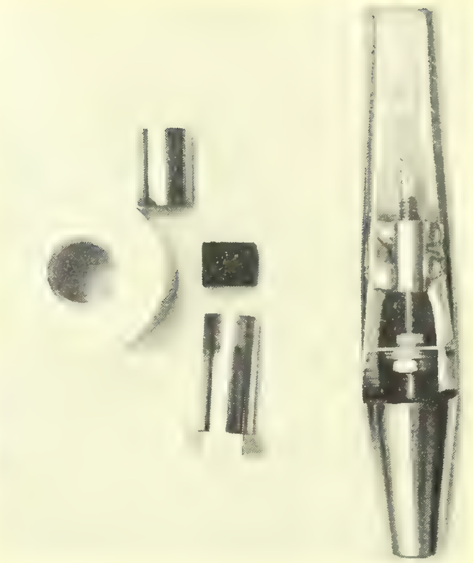

Figure 15.—Mold and arch punch.

Report

For report on this pellet abrasion test, record sample identification and history (aging, etc.), test temperature, pellet weight before and after abrasion. Calculate abrasion loss as weight and as a percent of original weight of pellet. When test results for duplicate pellets do not check within 0.010 gram or 10 percent, whichever is larger, two more pellets should be molded and tested. When pellets disintegrate completely before the end of the test, calculate from the elapsed time the number of revolutions to 100 percent abrasion loss. Also, report whether, in the operator's judgment, the pellet disintegrated because of brittleness or excessive softness and lack of cohesion.

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A Study of Viscosity-Graded Asphalt Cements

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

Reported by [1] J. YORK WELBORN, Principal Research Engineer, and EDWARD R. OGLIO and JOSEPH A. ZENEWITZ, Chemists, Materials Division

As part of a national effort to improve the quality of highway structures, the Bureau of Public Roads has initiated a comprehensive research program directed toward the development and use of fundamental knowledge to define the essential functional properties of asphalt and to recommend realistic tests and material requirements to specification writers. This article concerns a laboratory study of asphalt cements representing a broad range of sources that were collected by the Bureau of Public Roads to investigate the feasibility of grading and specifying asphalts by viscosity at 140° F. in lieu of penetration at 77° F. A secondary purpose for this article is to disseminate the test data to other researchers, who are using the same asphalts in their work.

Among other findings, the results indicate that when asphalt is graded by viscosity at 140° F. resultant materials will have more uniform within-grade consistency at temperatures of 140° F. and higher than materials obtained by use of the present system of grading asphalts by penetration at 77° F. However, the wide range in viscosity at temperatures of 77° F. and below recorded in tests during this research indicates a need for a requirement to control asphalt consistency at low temperatures. No evidence was noted during the tests reported here that viscosity grading caused any significant change in temperature susceptibility or resistance to hardening from changes previously noted for penetration graded asphalts.

Although some of the information reported here can be applied in the development of better specifications to define and control the essential functional properties of asphalt, the authors believe that additional research is needed to evaluate the binder rheology with that of paving mixtures, including the influence of mixture design, temperature, and aging. The authors recommend that this research be directed toward development of information from pavements in service and from carefully designed experiments in which the asphalts will be described on the basis of fundamental properties.

Introduction

ASPHALTIC materials are of prime economic importance in the construction and maintenance of the National highway system. Many thousands of miles of asphalt pavements—from low cost secondary to the highest type Interstate roads—have been constructed. An ever increasing demand is being made for better materials and construction methods that will provide higher quality and more durable pavements to carry modern and future traffic. A need also exists for a reduction in the cost of highway construction. Such goals can be met only by innovations in technology of structural design, construction practices, and maintenance operations. Better utilization of current materials and the development of new materials capable of producing lower cost, longer life, and superior performance will be required in the future.

In 1965 the Bureau of Public Roads promulgated a National Program of Research and Development to provide the knowledge of methods and materials to increase highway engineering productivity. As a beginning, the need for a better understanding of the properties of asphalt materials, especially those properties that affect the performance of pavements in service, is evident.

Thus, as part of the national effort, the Bureau of Public Roads has initiated a comprehensive research program directed toward the development and use of fundamental knowledge to define the essential functional properties of asphalt and to recommend realistic tests and material requirements to specification writers. Because of the greatly accelerated interest in asphalt research by the States, Federal Government, universities, industry, and other groups, it is essential that the national coordination of the overall effort be stressed. Such a program was outlined and discussed as part of a 3-day conference in Washington, D.C., April 7, 1965, on quality control and acceptance specifications (*1*) [2]. In the session on asphalt technology, representatives from the asphalt industry, the paving contractors, and the consumer interests expressed the need for: (1) better tests that will measure and control consistency, (2) tests that will predict durability in service, (3) specification requirements that will recognize variability that is the result of manufacturing, sampling, and testing, and (4) specifications that will provide requirements that will assure the proper balance of engineering properties.

To acquire the information desired, the Bureau of Public Roads is stressing the need for a coordinated research program for the development of instrumentation and techniques to measure and define the fundamental properties of asphalt binders that are related to rheology, durability, and chemistry, and to determine the relation of these properties to mixture design, asphalt-aggregate systems, and pavement performance.

The need for fundamental research on the properties of asphalt is not new and many reports and discussions have been written on different aspects of the subject. However, it was not until about 1960 that serious consideration actually was given to the fundamental aspects of the problem. Because consistency was believed to be of primary importance, the research effort was concentrated on the development of tests to measure viscosity in fundamental units and to determine the relation of these fundamental properties to mixture design and pavement performance.

In 1962 the Bureau of Public Roads published a report (*2*) speculating on the use of absolute and kinematic viscosity to control the consistency of asphaltic materials. In this article the possible advantages and disadvantages of such an approach for specification purposes were pointed out. Then, considerable evidence showed that the application of fundamental viscosity measurements to specifications for liquid asphalts of the cutback and slow curing types was practicable. Since then standard test methods have been developed and the adoption of specifications using these methods has become a reality. All major national specifications and a majority of the State highway departments now grade liquid asphalts by kinematic viscosity at 140° F. There is evidence that the grade limits may need some adjustment but the basic principle of using fundamental tests to measure consistency has proved workable.

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[1] Presented at the annual meeting of The Association of Asphalt Paving Technologists, Minneapolis, Minn., Feb. 1966.

[2] Italic numbers in parentheses indicate the references listed on page 41.

In 1963 a concerted effort was made to accelerate the study to determine the significance of using absolute and kinematic viscosity to measure the consistency and control the characteristics of asphalt cements at temperatures encountered in construction and in the resultant asphalt pavements in service. The application of fundamental viscosity measurements over such a range in temperatures presented some complex problems. Suitable test methods were needed to measure viscosity at temperatures below 100° F. Such methods would have to be reliable and simple to use. Also, asphalt cements from sources normally being used were known to differ greatly in viscosity-temperature susceptibility and to exhibit varying degrees of complex flow at low temperatures. The significance of these differences in terms of pavement performance would have to be determined before optimum specifications could be written. Continued research on these problems was supported by a recommendation of the Highway Research Board Special Committee No. 5 for Research Problems of Mutual Interest and Concern to Users and Producers of Asphaltic Materials. The major points in the Committee's recommendations were that instrumentation and test methods should be developed for measuring viscosity at low temperatures and that characteristics of asphalt cements at the low temperatures be studied.

Findings

The intent of the study reported here was to present and evaluate the physical properties of a series of asphalt cements prepared to meet a specification using viscosity grading at 140° F. Particular emphasis was given to the characteristics of the asphalts at temperatures associated with pavements in service. The principal findings are summarized as follows for the asphalts included in this study:

- Asphalt cements graded on the basis of viscosity at 140° F. have more uniform consistency within grade at temperatures above 140° F. than asphalts graded by penetration at 77° F.
- Viscosity grading at 140° F. provides a wider range in asphalt consistency at temperatures below 77° F. than is obtainable for asphalts controlled by penetration at 77° F. Thus, there is an apparent need for a specification requirement to control low-temperature properties of asphalts graded by viscosity at 140° F.
- The range in viscosity-temperature susceptibility between 140° and 275° F. is similar to the range for penetration graded asphalts.
- The sliding plate viscometer, using controlled rates of shear, is a satisfactory method for determining viscosities at low temperatures over a wide range of shear rates.
- Varying degrees of deviation from a straight line occurred in the viscosity-temperature relations below 140° F. when data were plotted on the Walther chart.
- Essentially straight line relations were shown when the log of the limiting viscosity and temperature in the relatively narrow range between 39.2° and 60° F. were plotted.
- Viscosity data obtained with two viscosity methods at several test temperatures and rates of shear, when plotted, superposed to a single flow diagram for each asphalt at a single test temperature. This superposition substantiates previous findings that the superpositioning technique can be used to evaluate rheological properties of viscosity-graded asphalts.
- A good correlation was obtained between log viscosity at 0.05 sec.$^{-1}$ rate of shear and log penetration at 60° and 77° F.
- A good correlation was obtained between shear susceptibility and log ductility at 60° F. on the original asphalts and thin-film residues, indicating the possibility of the use of a test for shear susceptibility in place of a ductility test.

The data presented in this report for viscosity-graded asphalts provide fundamental information that can be applied to the development of optimum specifications to define and control the essential functional properties of asphalt. However, the significance of some of the rheological properties of asphalts indicated by fundamental viscosity measurements must be studied further before requirements can be recommended for use in standard specifications. Research needs to be continued to evaluate the binder rheology with that of paving mixtures, including the influence of factors induced by mixture design, temperature, and aging. More knowledge is needed to correlate pavement performance adequately with asphalt properties measured by empirical tests presently in use and by tests related to the fundamental characteristics. This research should be directed toward development of information from pavements in service and from carefully designed experimental projects in which the asphalts used will be described on the basis of fundamental properties.

Study Specifications of The Asphalt Institute

One of the steps taken to accomplish the recommendations of the committee was the development of study specifications by The Asphalt Institute, College Park, Md., 1963. The requirements included in these specifications are set forth in table 1. The major element of the asphalt specifications is that of absolute viscosity measurements at 140° F. The primary advantage of such a specification is that the consistency of all asphaltic road binders would be graded at the temperature associated with maximum pavement temperature and the temperature used in some mixture design methods. It was also believed that such control would tend to eliminate some of the nonuniformity in asphalt cements and some of the variations in behavior encountered in construction and at high temperatures of pavement in service. Previous study (*3*) by the Bureau of Public Roads, using penetration grade asphalts from many different sources in paving mixtures made with crushed stone, gravel and sand, and sand alone, showed that compressive strength was dependent on the viscosity of the contained asphalt, as well as the type of aggregate used in the mixture. These findings supported the idea that good results could be obtained by grading asphalts on the basis of viscosity at 140° F., the approximate temperature at which the stability or instability of paving mixtures in the pavement is most critical.

In addition to the control of asphalt grades by viscosity at 140° F., The Asphalt Institute study specifications call for information for a minimum viscosity requirement at 275° F. and a provision to control hardening, using the ratio of viscosity at 140° F. after and before the thin-film oven test. Other proposed requirements cover ductility, flash point, and solubility in CCl_4 on the original asphalt.

Asphalts Studied

The viscosity-graded asphalts used in the study reported here were collected by the coordinated effort of The Asphalt Institute and the Bureau of Public Roads. The selection of the sources of production was based on the knowledge of the viscosity-temperature relationships of penetration-grade asphalts reported in previous studies of highway asphalts produced and used in the United States (*4*, *5*).

Samples, totalling 25 gallons, of each grade of asphalt were obtained directly from asphalt producers and are believed to have been a fair sample of total production in the United States. Asphalts of AC-5 and AC-10 grades produced to meet the viscosity ranges at 140° F. under The Asphalt Institute study specifications were obtained from 15 sources. The AC-20 and AC-40 grades were obtained from 14 and 4 of these sources, respectively.

Table 1.—Asphalt study specification requirements

Properties	Viscosity graded asphalts			
	AC-5	AC-10	AC-20	AC-40
Viscosity at:				
140° F. ____poises	500–750	1,000–1,500	2,000–3,000	4,000–6,000
275° F., centistokes	150+	200+	300+	400+
Ductility at:				
77° F.________cm		100+	100+	100+
60° F.________cm	60+			
Solubility in CCl_4, pct	99.5+	99.5+	99.5+	99.5+
Flash point, C.O.C.[1], °F	375+	425+	450+	450+
Thin-film oven test viscosity ratio[2]____________	5	5	5	

[1] Cleveland open cup test.

[2] Viscosity of residue at 140° F. ÷ viscosity of original asphalt at 140° F.

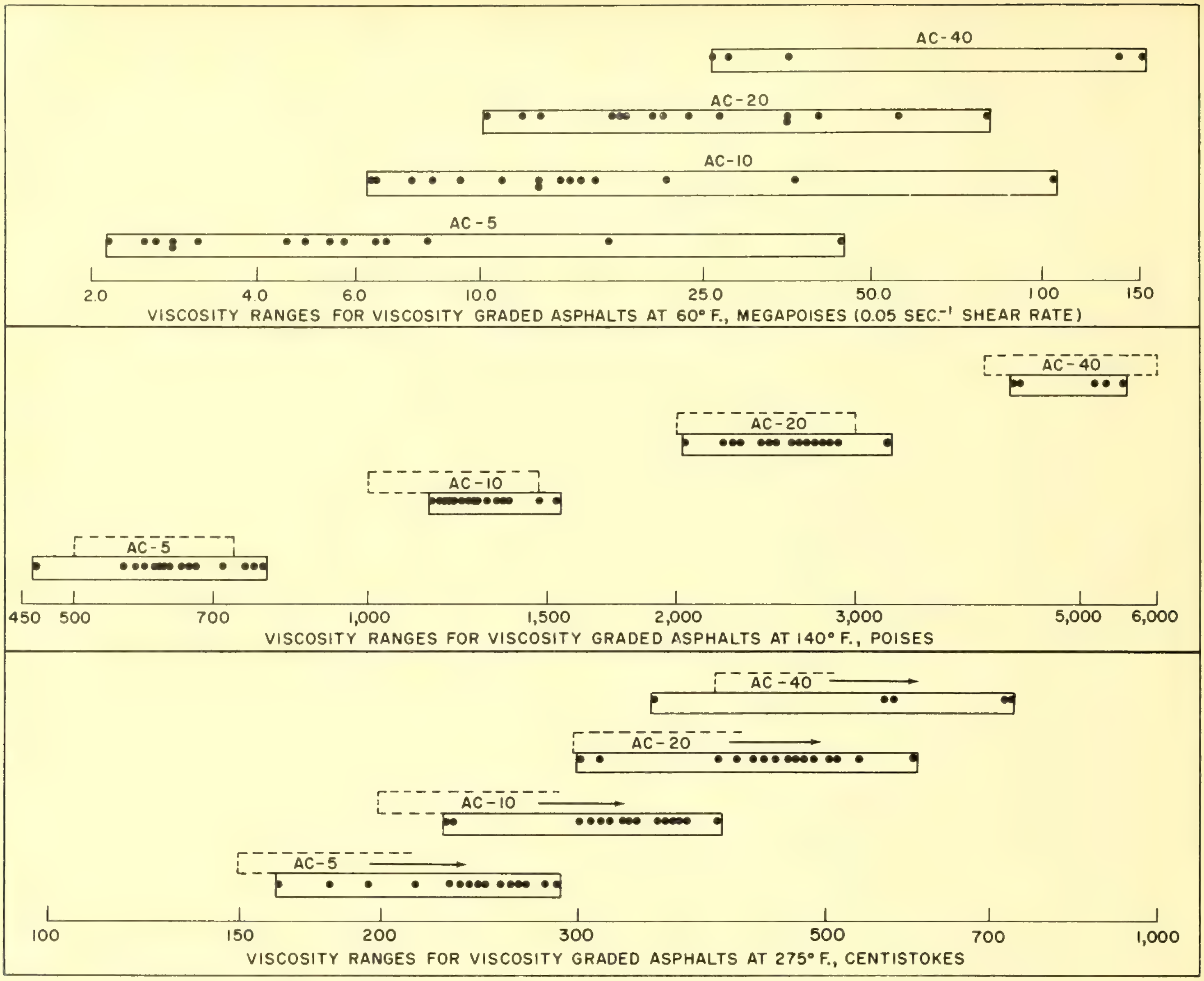


Figure 1.—Ranges and distributions of viscosity test results by viscosity grade of asphalt cements at different temperatures.

Asphalts from one other source that met the viscosity limits for AC-20 and AC-40 grades, submitted as penetration grade materials, also were included in the study. Some of the asphalts furnished were special products and, except for the source of the base products, do not necessarily represent commercial products based on present penetration grade specifications. They may not represent regular production that would be available under viscosity-grade specifications.

The samples were collected by the Bureau of Public Roads and were divided with The Asphalt Institute, which in turn provided samples for some of its company members. Public Roads in turn furnished samples to several State highway departments and to universities conducting research for the States. Some of the asphalts were or are being used in research projects initiated by the States and supported by Federal funds. Also, in accordance with the Public Roads effort to coordinate research efforts under the National program, the materials were made available to contractors in the National Cooperative Highway Research Program and to the Bureau of Mines for inclusion in their fundamental studies of asphalt.

Test Methods

Except for low temperature viscosity tests and a minor modification in the ductility tests, standard ASTM test procedures generally were used. For ductility tests, single specimens were used rather than the three prescribed specimens. Four different types of viscometers were used to cover adequately the temperature range over which viscosity determinations were made. The viscometers, applicable test method, and the test temperature at which they were used, are described in the following paragraphs.

- Zeitfuchs Cross-Arm Capillary Viscometer, ASTM Designation: D–2170–63T, "Tentative Method of Test for Kinematic Viscosity of Asphalts," 275° and 210° F.
- Cannon-Manning Viscometer, ASTM Designation: D–2171–63T, "Tentative Method of Test for Absolute Viscosity of Asphalts," 140° and 120° F.
- Sliding Plate Microviscometer, in accordance with the method described by Fink and Heithaus (*6*), 77° and 100° F.
- Sliding Plate Viscometer, in accordance with "Proposed Method of Test for Viscosity of Asphalt with a Sliding Plate Viscometer at Controlled Rates of Shear" (*7*) 60°, 45°, and 39.2° F. This viscometer and method, which were developed in the Public Roads laboratory by the authors, are described briefly later in this article.

The precision of the capillary and sliding plate microviscometer test methods have been established by usual interlaboratory round-robin tests and are given in the respective test methods. The precision of the sliding plate viscometer, in which controlled rates of shear are used, was estimated from tests made at 60° F. in the Public Roads laboratory. The

Table 2.—Statistical summary of precision data for repeatability of viscosity determinations at 60° F.

Asphalt code number and statistics	Replicates	Thickness of specimens	Viscosity, at shear rates of— 0.1 sec.⁻¹	0.05 sec.⁻¹	0.01 sec.⁻¹	0.005 sec.⁻¹	0.001 sec.⁻¹
	No.	*Microns*	*Megapoises*	*Megapoises*	*Megapoises*	*Megapoises*	*Megapoises*
C-5:							
Average	5	468.8	2.252	2.386			
Range	5	35	0.09	0.20			
Standard deviation	5	15	0.044	0.091			
Coefficient of variation ... pct	5	3.20	1.92	3.81			
B-10:							
Average	5	471.0	12.70	14.12	17.84	18.58	18.70
Range	5	157	1.1	0.1	1.7	1.3	1.3
Standard deviation	5	124	.418	0.050	0.634	0.605	0.660
Coefficient of variation ... pct	5	26.3	3.29	0.35	3.55	3.26	3.53
I-10:							
Average	5	452.2	14.50	15.56	16.82	16.98	16.98
Range	5	90	0.5	1.0	1.6	1.5	1.5
Standard deviation	5	35	0.187	0.403	0.657	0.605	0.605
Coefficient of variation ... pct	5	7.74	1.29	2.59	3.91	3.56	3.56
D-20:							
Average	5	478.6	15.58	17.75	21.74	22.28	22.40
Range	5	71	0.8	0.2	1.4	1.3	1.0
Standard deviation	5	30	.303	0.100	.630	0.668	0.547
Coefficient of variation ... pct	5	6.26	1.94	0.56	2.90	3.00	2.44
F-20:							
Average	5	488.4	12.80	15.64	24.40	28.80	31.35
Range	5	155	1.0	0.8	0.9	3.3	1.2
Standard deviation	5	131	0.400	0.320	0.424	1.31	0.900
Coefficient of variation ... pct	5	26.8	3.12	2.05	1.74	4.55	2.87
O-40:							
Average	5	564.4	119.0	156.2	212.2	215.4	215.4
Range	5	384	10	13	8	10	10
Standard deviation	5	162	4.30	4.60	4.38	4.98	4.98
Coefficient of variation ... pct	5	28.7	3.61	2.94	2.06	2.31	2.31
Pooled:							
Coefficient of variation ... pct			2.53	2.05	2.83	3.34	2.94
Repeatability ... do			7.34	5.95	8.23	9.72	8.56

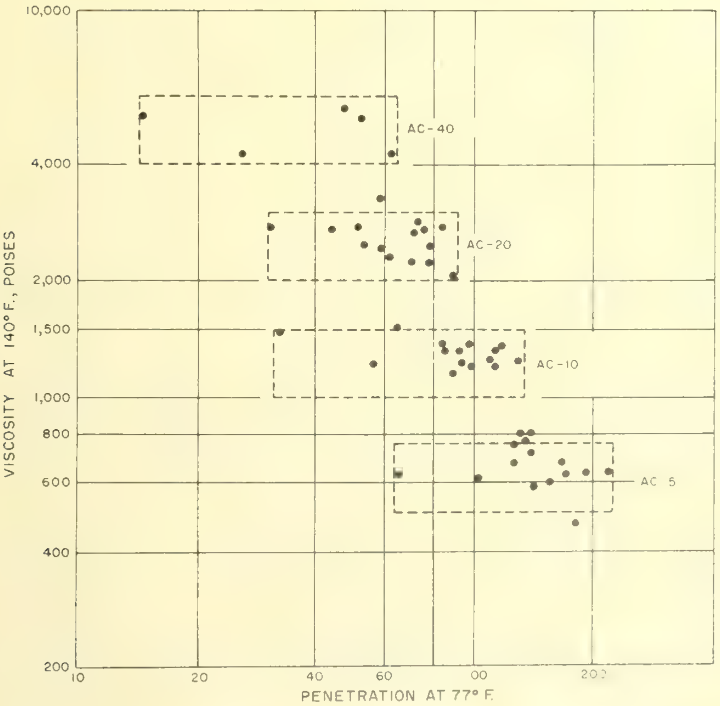


Figure 2.—Results of viscosity tests at 140° F. and penetration at 77° F. for the four viscosity graded asphalt cements.

Table 3.—Precision of viscosity test methods

Method or instrument	Repeatability,[1] 95-percent confidence level	Reproducibility,[2] 95-percent confidence level
	Pct. of mean	*Pct. of mean*
ASTM D 2170-63T	1.8	8.8
ASTM D 2171-63T	7	10
Microviscometer	13	26
Controlled shear rate[4]	8	

[1] Duplicate results obtained by the same operator.
[2] Single results from each of two laboratories.
[3] At 0.05 sec.$^{-1}$ shear rate, 77° F.
[4] Pooled data from table 2, 0.1 to 0.001 sec.$^{-1}$ shear rate, 60° F.

tests consisted of five replicate determinations made on different days by one operator on six of the asphalts selected to obtain three different levels of viscosity and three different levels of shear susceptibility. A statistical summary of the test results is given in table 2. For comparison, table 3 contains lists of the precisions for the four viscosity methods used in the study reported here. The test data are given in terms of the percentage of the average of two tests, and they reflect the manner of expressing test method precision used by the ASTM Committee D-4, namely, the maximum difference between two test results that may be expected at the 95-percent confidence level.

The precision for repeatability of the method, when controlled shear rates were used, was 8 percent and this is comparable to the precision obtained for the absolute viscosity method, ASTM D-2171-63T. However, this precision was based on tests made in one laboratory and may differ from that based on interlaboratory tests. Further work is needed to establish both repeatability and reproducibility for the low temperature tests involving the sliding plate and possibly other methods. Further work also is needed to develop more rapid methods. Results obtained late in the study reported in this article show that comparable results were obtained when the time of room conditioning was reduced from 1 to 1½ hours to 15 minutes and the time of bath conditioning was reduced from 25 minutes to 15 minutes, thus reducing total time of preparation and testing from 2 hours to 1 hour.

Test Results

The test results on the viscosity-graded asphalts are given in tables 4, 5, and 6. Two identification numbers are given, an asphalt code number and the corresponding Bureau of Public Roads laboratory number. The asphalt code number consists of a letter followed by a number. The letter serves to identify the refinery and the number identifies the viscosity grade. The laboratory number provides sample identification for other investigators and the cooperators who have included these asphalts in their studies.

Viscosity

In figure 1 the range and distribution of the viscosity data are summarized within each of the four study specification grades at the two test temperatures specified and at 60° F. and 0.05 sec.$^{-1}$ rate of shear. The boxes formed

Table 4.—Characteristics of viscosity-graded asphalt cements

Asphalt code number	BPR laboratory number	Viscosity		Penetration 100 g., 5 sec.			Ductility				Specific gravity, 77°/77° F.	Flash point	
							1 cm./min.	5 cm./min.					
		140° F.	275° F.	45° F.	60° F.	77° F.	45° F.	45° F.	60° F.	77° F.		C.O.C.[2]	P-M.[3]
		Poises	*Centistokes*				*Cm.*	*Cm.*	*Cm.*	*Cm.*		° *F.*	° *F.*
A-5	B-2908	754	257	17	42	125	126	31	215	130	1.004	600	500
B-5	B-2920	773	289	12	40	134	150+	31	250+	181	1.001	670	415
C-5	B-2958	662	264	32	68	180	150+	116	225	148	1.023	505	515
D-5	B-2962	642	231	28	70	192	150+	150+	250+	155	1.010	540	515
E-5	B-2974	627	246	6	16	64	3.5	3	16	152	0.999	645	345
F-5	B-3008	602	247	19	46	154	150+	59	250+	139	0.991	670	520
G-5	B-3012	669	282	22	60	175	150+	150+	250+	177	1.018	610	555
H-5	B-3028	459	195	26	65	191	171	71	250+	151	1.011	575	515
I-5	B-3037	586	240	13	40	139	175+	179	250+	172	1.022	640	585
J-5	B-3050	636	267	30	75	216	150+	150+	250+	154	1.021	460	460
K-5	B-3054	610	215	10	30	101	21	6	129	163	1.008	515	310
L-5	B-3058	632	267	23	58	169	150+	150+	250+	140	1.021	550	465
M-5	B-3108	791	238	15	43	136	175+	244	250+	167	1.011	515	460
N-5	B-3578	719	181	16	42	137	175+	150+	250+	205	1.015	465	455
O-5	B-3601	674	160	11	35	125	250+	250+	250+	171	1.011	580	525
A-10	B-2909	1,367	335	14	30	83	39	9	192	221	1.007	610	520
B-10	B-2921	1,152	339	11	28	88	29	8	243	250+	1.004	680	420
C-10	B-2959	1,352	401	20	45	115	116	22	177	157	1.031	530	485
D-10	B-2963	1,227	315	18	43	114	150+	150+	250+	250+	1.015	570	545
E-10	B-2975	1,484	355	4	9	32	(1)	(1)	5	215	1.005	670	605
F-10	B-3009	1,361	367	15	36	96	33	12	170	230	0.994	660	560
G-10	B-3013	1,209	376	15	37	112	150+	150+	250+	245	1.021	600	525
H-10	B-3029	1,208	309	14	35	97	91	16	250+	190	1.022	590	500
I-10	B-3036	1,268	333	9	26	84	141	11	250+	210	1.026	650	460
J-10	B-3051	1,255	374	18	46	128	150+	124	250+	210	1.028	480	455
K-10	B-3055	1,233	302	6	15	55	6	3	32	206	1.016	530	500
L-10	B-3099	1,257	362	16	39	108	84	33	250+	200	1.025	565	480
M-10	B-3109	1,317	322	10	28	91	175+	19	250+	250+	1.014	535	455
N-10	B-3579	1,208	227	10	27	94	250+	150+	250+	245	1.016	510	455
O-10	B-3602	1,529	233	5	18	64	250+	12	250+	250+	1.016	580	535
A-20	B-2910	2,340	417	10	22	61	9	9	47	210	1.012	600	510
B-20	B-2922	2,497	464	6	16	52	6	(1)	17	250+	1.007	655	430
C-20	B-2960	2,060	482	14	32	87	62	11	161	160	1.034	560	505
D-20	B-2964	2,696	452	10	25	70	175+	10	250+	241	1.021	590	555
F-20	B-3010	2,861	490	12	27	72	10	7	24	205	0.995	635	580
G-20	B-3014	2,261	506	11	26	76	175+	14	250+	250+	1.025	625	515
H-20	B-3030	3,286	467	8	20	57	11	5	88	250+	1.028	570	490
I-20	B-3035	2,786	606	6	16	50	8	4.5	250+	250+	1.034	640	475
J-20	B-3052	2,743	534	11	28	82	118	18	250+	250+	1.033	495	470
K-20	B-3056	2,684	430	3	11	30	3.5	3	10	250+	1.020	545	515
L-20	B-3060	2,741	513	12	27	74	45	8	250+	250+	1.030	585	480
M-20	B-3110	2,261	401	8	23	69	120	9	250+	250+	1.018	535	485
N-20	B-3580	2,461	314	6	16	57	243	4	250+	250+	1.022	490	470
O-20	B-3603	2,729	301	4	12	43	72	(1)	250+	250+	1.021	585	505
P-20	B-3039	2,485	442	12	30	77	11	7	47	250+	1.004	635	450
J-40	B-3053	5,266	745	7	20	51	13	6.5	193	250+	1.035	510	480
K-40	B-3057	5,217	574	2	6	14	(1)	(1)	3	231	1.025	570	515
L-40	B-3061	5,537	742	6	17	46	8	5	58	250+	1.038	615	485
O-40	B-3604	4,286	353	2	7	25	(1)	(1)	35	250+	1.025	620	555
P-40	B-3040	4,286	567	10	23	60	7	4	18	230	1.006	665	515

[1] Fracture.
[2] Cleveland open cup test.
[3] Pensky Martens test.

by the solid lines show the ranges in viscosities for each grade. The dots within the boxes represent the viscosities obtained for each asphalt and show the distribution of results obtained for each viscosity grade. The broken line boxes show the study specification requirements at 140° and 275° F. for each grade. The figure shows that viscosity test results outside the study specification limits at 140° F. were obtained on 6 of the 50 asphalts: 5 results exceeded the maximum limits and one was below the minimum limit. Some of the asphalts furnished for the study represented special production and the noncompliance with specification limits may not be typical of regular production. However, caution should be used in adjusting specification limits so that production and testing variability and the criticalness of the requirements in use and performance will be recognized. The Bureau of Public Roads is concerned about such adjustments and has recommended that, when possible, specification requirements be based on targets having optimum tolerances. For example, viscosity requirements comparable to those in the study specifications would be specified as 600±150, 1,200±300, 2,400±600, and 4,800±1,200. This would provide a general recognition that materials should be furnished in the center of the grades and also would provide a better basis for statistical evaluation.

Also, figure 1 data show that test results for viscosity at 275° F. were above the minimum specified for each grade, except for asphalt sample O-40. The results for other grades from the same source, as well as those from producer N were near the minimum requirement. The location of viscosity results at 275° F. for the different grades of asphalt from the same source show that more attention should be given to setting the viscosity limits to account for a change in grade of asphalt. For example, the viscosity of asphalt O-40 is below the minimum limit; the viscosity of asphalt O-20 is slightly above the minimum limit, and the viscosities of asphalts O-10 and O-5 are appreciably above the minimum limit. Setting limits for other requirements applicable to specifications for two or more asphalt grades also should be considered.

As shown by comparison of the ranges in viscosity at different temperatures in figure 1, on the basis of grading at 140° F., extensive overlapping occurred in the results obtained at 60° F. (0.05 sec.$^{-1}$ shear rate) and 275° F. on asphalts of the four grades. As is well known, overlapping of consistency measurements at temperatures other than the grading temperature also occurs in the penetration grading system presently in use. This overlapping is caused by the differences in temperature susceptibility of asphalts. In the temperature interval of 140° F. to 275° F., temperature susceptibilities ranged from about −3.4 to −3.9 for the asphalts having the lowest susceptibility and the highest susceptibility, respectively, within each grade. These results were determined from the Walther relation, as follows:

$$\text{Viscosity-temperature susceptibility} = \frac{\log\log \eta_2 - \log\log \eta_1}{\log T_2 - \log T_1}$$

Where,

η_1 = Viscosity in centipoises at 275° F.
η_2 = Viscosity in centipoises at 140° F.
$T_1 = (275+459)$
$T_2 = (140+459)$

Previous studies have shown that asphalts supplied under the present penetration grading system range in susceptibility from about −3.1 to about −3.9, which is comparable to the range determined in the study for the viscosity-graded asphalts.

Within-grade uniformity

As indicated previously, one of the advantages to be expected when grading asphalt cements on the basis of absolute viscosity at 140° F. is that better uniformity in within-grade consistency would be obtained at construction temperature than is now being obtained by penetration grading at 77° F. This expectation is a logical consequence of shifting the grading control point from 77° F. to 140° F. That this expectation was reasonable can be seen from an examination of the results in figure 1 showing about a twofold spread in within-grade viscosity at 275° F. This is less than the threefold spread in previous tests for penetration graded asphalts (*5*).

Penetration for Viscosity-Graded Asphalts

Much of the early work done to develop specifications based on viscosity grading at 140° F. was concerned with the effect of such a system on penetration (*2*). Accordingly, estimates were made of the ranges in penetration at 77° F. that could be expected by specifying asphalt cements by viscosity at

40° F. with no other requirements to control temperature susceptibility. These estimates were based on extrapolations that assumed a straight line relation between viscosity and temperature over the full 77° F. to 275° F. ange on Walther charts and slopes of −3.1 and −3.9 for the lowest and highest susceptibilities. The penetration ranges estimated on his basis for the AC-5, AC-10, AC-20, and AC-40 asphalt grades were 140 to 400, 87 to 250, 60 to 150, and 34 to 85. As shown in figure 2 and table 4 penetration at 77° F. ranged from 64 to 216, 32 to 128, 30 to 93, and 14 to 60 for the asphalts included in the our grades used for the study reported here. These results are appreciably lower than had been estimated.

One reason for the discrepancy between the penetrations actually obtained and those originally estimated can be seen in figure 3, which shows plots of the viscosity data for our asphalts on a Walther-type chart over he temperature range 39.2° F. to 275° F. These asphalts were selected because they represent the extremes in viscosity below 77° F. at 0.05 sec.$^{-1}$ for the AC-5 and AC-10 grades. None of the four asphalts produced a straight line over the full temperature range covered and the curves show distinct departure from linearity in the 39.2° to 77° F. temperature range. The actual curves at the lower temperatures are above the projection of the straight lines drawn between the 275° F. and the 140° F. points. This departure, or offset, occurred in most of the asphalts and the amount of offset varied according to the asphalt in relation to its source. Sometimes, such as for the J asphalts marked in figure 3, the offset was relatively slight. For a few asphalts, such as the E asphalts, the offset was comparatively large. For other asphalts the amount of offset was somewhere between these extremes. Changes in rheological responses in this area have been noted by other investigators (*8*, *9*) on penetration-graded asphalts. They theorized that the softening point is a region of transition where asphalts change rapidly in physical structure, which causes changes in the asphalts temperature susceptibility and rheological character.

In previous discussion (*2*) of the feasibility of using viscosity at 140° F. for grading asphalt cements, it was pointed out that the result of control at one temperature could possibly be greater differences between materials of the same grade than occur with use of the present penetration grading and controls. It was also indicated that control at two or more temperatures should be considered before viscosity grading is adopted. The range in results shown in figure 1 for viscosity at 60° F. supports these contentions. The ranges in viscosity for each grade are extremely large at this temperature and it is evident that low temperature requirements are necessary. However, continued research is necessary to establish the optimum limits for such a requirement. These should be based on fundamental properties of mixtures and closely related to performance in layered pavement systems, including the effects of aging.

Table 5.—Characteristics of residues from thin-film oven test

Asphalt code number	Change in weight	Tests on residue										Retained penetration	Viscosity ratio at		Shear susceptibility, 60° F.[1]
		Penetration, 77° F.	Ductility, 5 cm./min.		Viscosity		Viscosity at 60° F., at shear rates of								
			77° F.	60° F.	140° F.	275° F.	0.001 sec.$^{-1}$	0.005 sec.$^{-1}$	0.01 sec.$^{-1}$	0.05 sec.$^{-1}$	0.10 sec.$^{-1}$		140° F.	275° F.	
	Pct.		*Cm.*	*Cm.*	*Poises*	*Centistokes*	*Megapoises*	*Megapoises*	*Megapoises*	*Megapoises*	*Megapoises*	*Pct.*			
A-5	+0.02	74	185	50	1,943	371	28	28	27	18	14	59.2	2.58	1.44	0.27
B-5	−.06	77	216	63	1,743	390	21	21	21	15	13	55.2	2.25	1.35	.21
C-5	−.17	101	220	35	1,882	405	18	18	12	11	10	56.1	2.84	1.53	.23
D-5	−.03	112	220+	250+	1,508	334	15	13	12	9	8	58.3	2.35	1.45	.15
E-5	−.09	38	68	4	1,709	347	320	180	127	56	39	59.4	2.73	1.41	.51
F-5	+.12	90	230	73	1,504	347	17	17	16	11	10	58.4	2.50	1.40	.25
G-5	+.10	108	228	243	1,157	353	10	10	10	8	7.5	65.1	1.73	1.25	.13
H-5	−.05	92	225	85	1,532	316	27	27	24	18	15	48.2	3.34	1.62	.22
I-5	+.07	96	250+	250+	1,027	301	12	12	12	11	11	69.1	1.75	1.25	.02
J-5	−.79	103	225+	140	2,212	474	8	8.0	7.6	6.7	6.3	47.7	3.48	1.78	.08
K-5	−.29	54	193	13	1,514	297	92	69	57	33	26	53.5	2.48	1.38	.34
L-5	−.22	115	220+	141	1,443	368	12	12	11	9	8	68.0	2.28	1.38	.12
M-5	−.57	74	234	168	2,092	364	30	30	28	22	19	54.4	2.64	1.53	.17
N-5	−.87	72	250+	250+	1,904	293	44	39	37	32	30	52.6	2.65	1.62	.09
O-5	+.07	94	250+	250+	999	191	11	11	11	11	11	75.2	1.48	1.19	.00
A-10	+.01	51	201	11	3,908	497	84	73	63	35	27	61.4	2.86	1.48	.37
B-10	−.11	54	250+	19	3,054	506	75	61	51	30	24	61.4	2.65	1.49	.32
C-10	−.22	70	170	23	4,039	634	35	34	32	21	17	60.9	2.99	1.58	.28
D-10	+.04	71	250+	94	3,135	465	33	29	26	19	16	62.3	2.56	1.48	.21
E-10	+.10	23	24	3	3,735	490	1,140	520	353	141	95	70.8	2.52	1.38	.57
F-10	+.10	61	213	10	4,089	508	48	44	38	23	17	63.5	3.00	1.38	.36
G-10	+.04	77	250+	191	2,316	493	25	25	25	20	18	68.8	1.92	1.31	.15
H-10	+.03	60	250+	15	3,497	463	58	56	50	30	24	61.9	2.89	1.50	.32
I-10	+.11	56	250+	118	2,360	441	37	37	36	29	27	66.3	1.86	1.32	.14
J-10	−.69	75	220+	61	3,826	634	32	30	27	21	18	58.6	3.05	1.70	.17
K-10	−.12	35	180	7	2,568	397	206	161	130	67	50	63.6	2.08	1.31	.42
L-10	−.15	70	250+	32	2,954	509	36	35	32	22	19	64.8	2.35	1.41	.24
M-10	−.31	55	250+	52	3,397	486	59	58	53	37	32	60.4	2.58	1.51	.22
N-10	−.68	53	250+	250+	2,971	350	50	50	48	42	39	56.4	2.46	1.54	.09
O-10	+.08	51	250+	250+	2,235	275	41	41	40	38	37	79.7	1.46	1.18	.04
A-20	−.02	39	148	6	7,047	640	176	138	106	55	42	63.9	3.01	1.77	.40
B-20	−.05	34	156	6	7,038	678	285	182	136	69	52	65.4	2.82	1.46	.42
C-20	−.11	56	159	15	5,479	749	75	62	53	32	26	64.4	2.66	1.55	.31
D-20	+.09	47	250+	35	6,311	649	71	67	62	37	29	67.1	2.34	1.44	.36
F-20	+.12	50	102	8	9,293	735	95	68	55	29	22	69.4	3.25	1.50	.41
G-20	−.01	53	250+	61	4,331	671	50	50	47	36	32	69.7	1.92	1.33	.17
H-20	.00	37	168	8	9,703	750	182	143	113	56	41	64.9	2.95	1.61	.46
I-20	+.11	33	225+	21	5,404	624	140	131	117	68	58	66.0	1.94	1.03	.31
J-20	−.51	50	250+	29	7,853	903	63	60	54	37	31	61.0	2.86	1.69	.25
K-20	−.05	24	137	4	5,300	564	590	359	265	115	82	80.0	1.97	1.31	.51
L-20	−.12	45	250	13	7,153	750	94	86	74	43	34	60.8	2.61	1.46	.33
M-20	−.24	41	213	21	6,424	637	108	102	88	57	47	59.4	2.84	1.59	.28
N-20	−.45	32	250+	150	5,627	479	115	115	113	84	70	56.1	2.29	1.53	.25
O-20	+.08	32	250+	37	4,210	365	170	170	168	115	95	74.4	1.54	1.21	.28
P-20	−.04	49	73	8	11,046	812	122	84	66	34	25	63.6	4.45	1.84	.42
J-40	−.46	29	119	7	16,143	1,278	135	123	107	63	50	56.9	3.07	1.72	.33
K-40	−.07	9	49	0	10,451	761	1,530	780	535	210	139	64.3	2.00	1.33	.59
L-40	−.06	28	250+	6	14,539	1,055	277	185	146	81	63	60.9	2.63	1.42	.37
O-40	+.09	19	250+	0	5,786	515	400	390	342	205	157	76.0	1.35	1.46	.40
P-40	+.06	43	72	6	14,267	900	182	120	90	41	30	71.7	3.33	1.59	.48

[1] Tangent of viscosity—rate of shear curve between 0.05 sec.$^{-1}$ and 0.10 sec.$^{-1}$

Limiting Viscosity

Other researchers have used limiting or initial viscosity in studying the rheological behavior of asphalts. This is the viscosity obtained when the material is behaving in a Newtonian fashion and is independent of the rate of shear. It occurs in asphalts at low shear stresses or low rates of shear and the viscosities obtained are higher than those in the shear-dependent or non-Newtonian region. Limiting viscosity was determined for all of the asphalts included in the study reported here and are given in table 7. The viscosity-temperature relations are shown in figure 4 for the same asphalts as those depicted in figure 3, but limiting viscosity rather than apparent viscosity at 0.05 sec.$^{-1}$ shear rate is plotted against temperature on a Walther-type chart. Comparison of data in figures 3 and 4 will show the difference between limiting viscosity and viscosity determined in the shear-dependent region. The curves in figure 4 are straighter and the asphalt viscosities are higher than shown by curves in figure 3.

To explore the low temperature properties of the asphalts the limiting viscosities were plotted using the ordinates of log viscosity in poises and temperature in degrees Fahrenheit. This relation for AC-10 grade asphalts is represented by essentially straight lines in figure 5. Similar relations existed for the other grades. Recognizing that the temperature range of 39.2° to 60° F. is relatively narrow, these relations are possibilities that should be considered in further evaluation of both the level of viscosity and temperature susceptibility of asphalt.

Viscosity Data Reduced to Master Curves

Because the microviscometer and the controlled rate of shear viscometer were used to determine viscosities at the temperatures below 140° F., study was made to determine whether the two instruments gave concordant results and also to indicate the general rheological character of the asphalts as affected by temperature and shear rate. The asphalts selected for the special study were A-5, E-5, B-10, and I-10. The viscosity results obtained at different rates of shear at test temperatures of 39.2°, 45°, 60°, 77°, 100°, and 120° F. are given in table 8.

All four asphalts produced flow diagrams as illustrated in figure 6 for asphalt A-5. The curves are similar to those obtained by Brodnyan (*10*) with penetration-graded asphalts and subsequently by others. It is interesting that the curves approach zero slope at the lower shear rates. As mentioned before, viscosity in this area is the limiting viscosity and it is independent of rate of shear. Curves for each of the four asphalts were superposed by the method described in reference (*11*) to a single master curve at a reference temperature of 60° F., as shown in figure 7. The amount of horizontal shifts (a_T) required to effect the smooth, reduced curves shown in the figure are given in table 9 and shown in figure 8. As will be noted, these factors permit obtaining viscosity at a single temperature over a much wider range of shear rates, approximately 6 decades in figure 7, than is now practicable with present viscometers. Furthermore the factors are considered (*10*) to be an indication of the temperature dependence of limiting viscosity and, as such, are useful in comparing asphalts as to temperature susceptibility.

The close conformance of the data points to the master curves in figure 7 indicates that the viscosities measured by the controlled shear rate viscometer at 39.2°, 45°, and 60° F. are substantially in agreement with those measured by the standard microviscometer at 77° and 100° F. The general shape of the curves also indicates that at these test temperatures the viscosity-graded asphalts appear to be similar, but the level of viscosity and the influence of this difference on the rheological properties of paving mixtures before and after aging needs further study.

Relation of Viscosity and Empirical Tests

The primary interest in studying the use of fundamental properties of asphalts is to determine whether they can be related to the fundamental properties of paving mixtures and whether they will ultimately provide a better basis for improved specifications. Also, of interest is a comparison of the results of fundamental properties to the characteristics measured by conventional empirical methods. So a part of the study reported here was devoted to examining the relation of viscosity data to penetration, ductility, and durability.

Penetration

The relation of penetration to viscosity detailed in tables 4 and 6, at 60° and 77° F., is shown in figures 9 and 10, respectively. As shown in the figures, good correlations of penetration and viscosity were obtained at

Table 6.—Viscosity at different rates of shear and temperatures

Asphalt code number	Viscosity, megapoises at shear rate and temperature of—																Shear susceptibility [1] at temperatures of—		
	0.1 sec.$^{-1}$			0.05 sec.$^{-1}$				0.01 sec.$^{-1}$			0.005 sec.$^{-1}$			0.001 sec.$^{-1}$					
	39.2° F.	45° F.	60° F.	39.2° F.	45° F.	60° F.	77° F.	39.2° F.	45° F.	60° F.	39.2° F.	45° F.	60° F.	39.2° F.	45° F.	60° F.	39.2° F.	45° F.	60° F.
A-5	68	38	6.4	87	45	6.7	0.46	153	67	7.6	196	79	8.0	250	96	[2] 9.0	0.36	0.25	0.08
B-5	62	46	7.2	84	56	8.0	.81	171	91	9.7	209	111	[2] 9.7	343	147	[2] 9.7	.44	.30	.14
C-5	25	18	2.5	31	21	2.6	.48	49	30	3.0	56	30	[2] 3.2	64	30	[2] 3.6	.29	.26	.08
D-5	26	17	2.5	30	19	2.5	.36	42	26	2.5	46	28	[2] 2.5	51	[2] 32	[2] 2.5	.21	.18	.00
E-5	197	129	33.9	298	184	44.0	3.50	775	414	82.5	1,180	585	103.0	3,100	1,300	137.0	.59	.50	.39
F-5	37	23	4.2	52	29	4.5	1.12	103	51	5.5	131	60	[2] 6.0	177	[2] 62	[2] 7.4	.47	.35	.12
G-5	37	23	2.8	45	25	2.8	.32	68	27	2.9	70	27	[2] 2.9	70	[2] 27	[2] 3.0	.28	.11	.02
H-5	30	14	2.4	37	18	2.8	.52	60	27	3.0	70	29	[2] 3.0	70	29	[2] 3.0	.30	.29	.23
I-5	79	48	6.2	102	54	6.5	.49	166	76	7.0	194	82	7.0	232	86	7.0	.34	.23	.06
J-5	33	16	2.2	40	18	2.2	.29	56	22	2.3	60	25	[2] 2.3	[2] 64	26	[2] 2.3	.24	.15	.04
K-5	92	72	14.0	130	96	16.8	.87	290	195	25.5	410	264	28.0	700	410	29.5	.50	.43	.27
L-5	28	21	3.1	35	23	3.1	.32	50	28	3.1	54	30	[2] 3.1	[2] 65	30	[2] 3.1	.31	.12	.00
M-5	70	40	5.5	82	46	5.6	.57	119	62	6.0	142	66	6.2	150	66	[2] 6.4	.23	.19	.04
N-5	90	42	4.8	105	48	4.8	.42	146	58	4.8	146	58	[2] 4.8	146	58	[2] 4.8	.21	.15	.00
O-5	126	67	5.3	153	74	5.3	.46	239	80	5.3	239	80	[2] 5.3	239	80	[2] 5.3	.29	.16	.00
A-10	100	58	14.4	130	74	16.0	1.49	247	134	20.2	323	170	21.5	540	227	[2] 22.3	.39	.36	.16
B-10	141	78	13.1	188	99	15.1	.96	370	174	19.2	495	215	19.4	920	250	[2] 20.0	.42	.35	.25
C-10	47	30	7.6	59	35	8.1	1.08	99	54	9.2	120	60	9.8	148	73	[2] 11.0	.32	.26	.08
D-10	62	36	6.4	78	42	6.5	.75	129	57	6.8	147	60	7.0	170	65	[2] 7.2	.31	.21	.02
E-10	310	268	78.0	487	360	109.0	13.5	1,400	990	237.0	2,200	1,530	330.0	5,770	3,850	625.0	.66	.63	.48
F-10	79	42	11.0	103	54	12.7	1.58	203	96	17.8	272	124	19.7	385	180	[2] 22.2	.41	.36	.21
G-10	81	44	7.5	106	50	7.5	.95	161	65	7.5	181	70	7.5	217	82	[2] 7.5	.32	.17	.00
H-10	88	52	11.1	105	65	12.7	1.14	230	112	13.4	310	137	13.8	470	174	[2] 14.7	.43	.34	.19
I-10	161	94	12.7	215	117	13.8	1.35	412	186	14.5	545	215	15.0	770	250	[1] 15.8	.42	.29	.09
J-10	56	33	6.4	68	37	6.4	72	103	46	6.4	123	49	[2] 6.4	145	50	[2] 6.4	.18	.15	.00
K-10	188	113	28.4	293	165	36.5	4.08	815	395	63.0	1,270	575	73.5	3,100	1,080	90.0	.64	.54	.35
L-10	66	42	8.4	85	52	9.1	1.08	147	85	12.0	186	94	13.3	220	94	[2] 13.3	.34	.30	.14
M-10	101	68	13.7	135	90	14.7	1.53	267	150	17.7	327	183	18.6	395	215	[2] 19.0	.43	.35	.11
N-10	155	86	10.5	203	94	10.8	95	333	109	12.0	360	109	12.6	380	109	[2] 13.0	.40	.14	.06
O-10	243	150	21.5	341	190	21.5	1.58	715	293	21.5	849	305	21.5	950	314	[2] 21.5	.50	.36	.00
A-20	124	70	17.7	177	101	21.5	3.40	400	233	32.2	560	317	35.0	975	405	[2] 35.0	.51	.53	.28
B-20	204	133	28.8	313	190	36.0	6.99	860	435	57.5	1,250	625	67.0	2,740	1,225	69.7	.62	.51	.33
C-20	79	44	9.3	106	57	10.5	1.41	209	101	13.1	272	123	13.5	419	153	[2] 13.7	.42	.37	.16
D-20	117	84	18.4	158	112	20.6	2.25	320	210	26.0	423	255	27.3	630	295	[2] 28.5	.44	.40	.16
F-20	90	44	14.8	127	62	18.5	2.49	278	132	30.6	385	183	36.1	760	345	38.0	.49	.47	.38
G-20	133	68	12.4	176	90	13.0	2.14	325	158	14.2	425	179	15.0	630	200	[2] 16.9	.39	.39	.07
H-20	148	93	21.9	220	131	27.2	3.68	555	283	42.0	780	363	46.1	1,620	600	[2] 48.0	.58	.49	.31
I-20	217	160	33.5	330	219	41.0	6.20	850	460	58.1	1,210	595	63.0	2,190	870	[2] 67.0	.59	.46	.28
J-20	108	67	11.2	143	86	12.1	1.73	287	143	14.3	360	168	15.0	460	202	[2] 15.4	.42	.35	.14
K-20	[2] 320	262	57.5	[2] 495	394	82.0	12.5	1,400	1,025	185.0	2,170	1,550	233.0	6,150	3,460	300.0	.64	.59	.50
L-20	110	73	18.9	149	94	23.8	2.93	300	185	30.1	387	232	32.4	465	238	[2] 32.7	.44	.41	.20
M-20	117	82	16.4	181	110	18.3	3.22	430	213	24.0	575	277	26.4	860	340	[2] 26.5	.54	.41	.17
N-20	273	160	34.6	375	200	36.0	2.52	775	320	38.5	1,040	339	39.8	1,300	350	[2] 41.5	.45	.31	.05
O-20	342	267	57.0	535	375	57.3	3.90	1,470	800	57.8	2,040	1,000	58.0	3,000	1,180	58.7	.65	.49	.01
P-20	89	45	14.7	126	62	17.7	3.18	282	136	27.9	400	178	33.7	700	280	[2] 44.0	.50	.48	.28
J-40	171	100	24.0	238	133	26.7	4.36	520	257	34.5	690	317	37.5	1,120	400	42.0	.48	.42	.16
K-40	426	297	96.0	710	505	138.0	27.7	2,279	1,630	323.0	3,571	2,700	435.0	10,445	6,900	690.0	.73	.73	.53
L-40	180	120	28.7	269	165	36.2	7.25	656	344	58.0	929	460	63.5	1,700	695	65.0	.56	.46	.32
O-40	[2] 500	365	125.0	[2] 890	520	154.0	12.4	1,640	1,170	217.0	2,000	1,650	217.0	2,740	2,600	220.0	.60	.51	.35
P-40	105	77	21.6	153	107	28.2	3.15	372	230	51.2	550	322	62.0	1,090	670	79.0	55	.48	.46

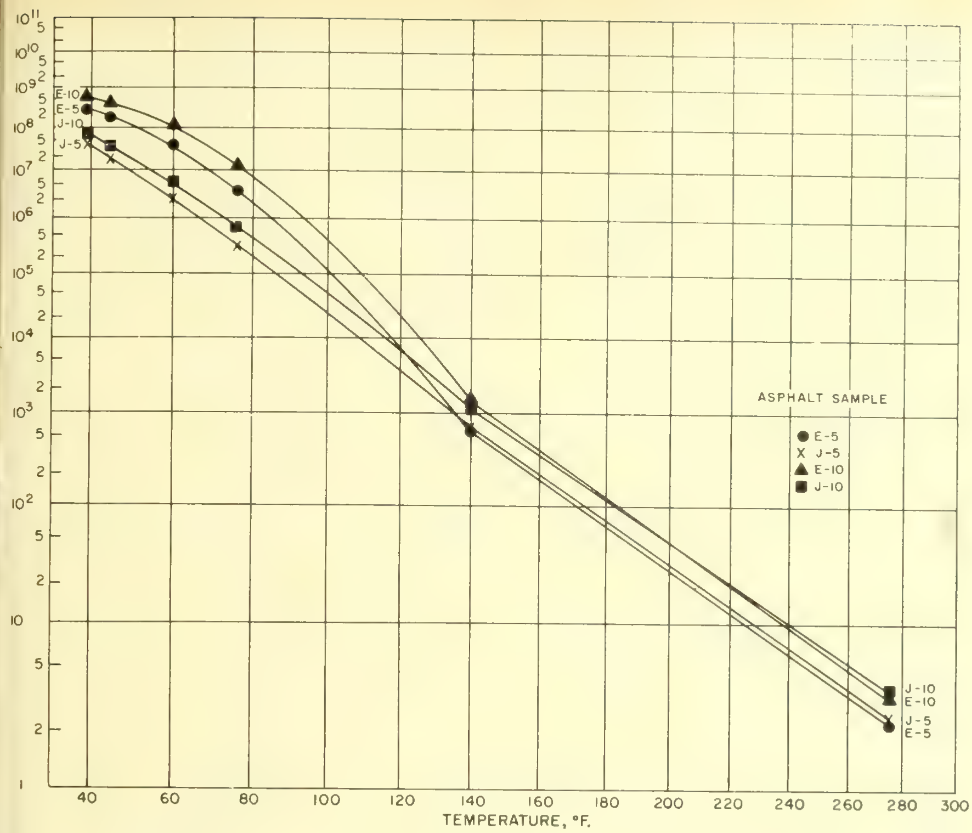
[1] Tangent of viscosity—rate of shear curve between 0.05 sec.$^{-1}$ and 0.1 sec.$^{-1}$.

[2] Values obtained by extrapolation from viscosity—rate of shear curve.

Table 7.—Limiting viscosities of asphalt cements, by viscosity grades

Asphalt code	Limiting viscosities, megapoises											
	AC-5			AC-10			AC-20			AC-40		
	39.2° F.	45° F.	60° F.	39.2° F.	34° F.	60° F.	39.2° F.	45° F.	60° F.	39.2° F.	45° F.	60° F.
A	250	97.0	9.50	[1] 625	240	23.0	[1] 1,300	[1] 510	35.0	...	...	...
B	[1] 400	150	9.70	[1] 730	250	20.0	[1] 3,900	[1] 1,250	69.0	...	...	...
C	64.0	30.5	4.00	150	77.0	11.3	[1] 435	153	13.6	...		...
D	54.0	34.0	2.50	170	67.0	7.40	650	290	28.6	...		...
E	[1] 15,000	[1] 4,400	137	[1] 36,000	[1] 12,000	[1] 860	...	...				...
F	180	62.0	4.30	[1] 450	[1] 210	23.0	[1] 940	[1] 360	38.0	...		...
G	70.0	27.0	3.00	225	87.0	7.50	630	210	17.5		...	...
H	70.0	29.0	3.00	[1] 500	175	15.0	[1] 2,100	[1] 630	48.5		...	...
I	235	85.0	7.00	[1] 800	250	16.0	[1] 2,700	[1] 920	67.0		...	...
J	64.0	25.7	2.30	130	50.5	5.50	470	202	15.5	[1] 1,160	420	42.0
K	[1] 1,090	[1] 410	30.0	[1] 4,000	[1] 1,220	[1] 93.0	[1] 15,000	[1] 4,500	[1] 310	[1] 21,400	[1] 8,200	[1] 720
L	70.0	29.5	3.15	222	93.6	12.0	470	238	33.0	[1] 2,150	[1] 730	64.0
M	155	66.0	6.80	520	215	19.2	900	350	26.5	...		...
N	146	60.0	4.93	380	130	13.1	1,290	450	41.5	...		...
O	239	80.0	5.32	950	315	22.0	[1] 3,200	1,200	58.7	[1] 6,500	[1] 2,700	240
P	...	...	...	...	...	...	[1] 700	[1] 360	[1] 45.0	[1] 2,600	[1] 1,020	[1] 79.5

[1] Values obtained by extrapolation from viscosity—rate of shear curve.



igure 3.—Viscosity (0.05 sec.$^{-1}$ shear rate) and temperature relation for selected asphalt cements.

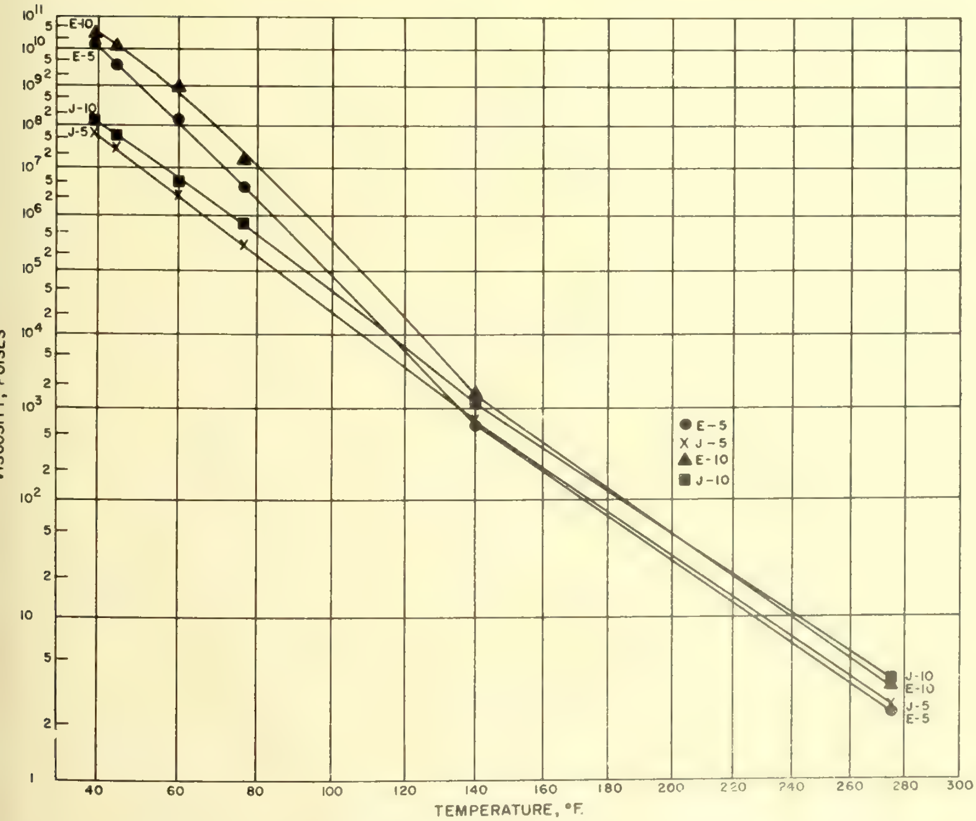


Figure 4.—Limiting viscosity and temperature relation for selected asphalt cements.

both test temperatures, but the correlation obtained at the 60° F. test temperature seems to have been somewhat better than that obtained at 77° F. The equations derived by the method of least squares for the relation between log viscosity and log penetration at each temperature are:

At 60° F.,

$$\log \text{viscosity (megapoises)} = 3.54 - 1.67 \log \text{penetration}.$$

At 77° F.,

$$\log \text{viscosity (megapoises)} = 3.86 - 1.89 \log \text{penetration}.$$

Ductility

The value of a ductility requirement in specifications for asphalts has been the subject of debate among asphalt technologists since its introduction in the early part of this century. Some technologists are of the opinion that ductility, under the present standard test method, is of little value as an indicator of asphalt quality. Others believe that the ductile properties of asphalt give an asphalt pavement its quality of flexibility—the ability to conform to moderate deflections or changes in supporting layers without permanent cracking or disintegration. Some believe that ductility is related to stickiness or ability to adhere to aggregate and other surfaces. Regardless of the merits of the various arguments, some studies (*12, 13*) have related ductility to pavement performance.

The ductility requirements proposed in the study specifications were a minimum of 60 cm. at 60° F. for grade AC-5 and a minimum of 100 cm. at 77° F. for grades AC-10, AC-20, and AC-40, respectively. The results of ductility tests, table 4, at these temperatures show that, except for asphalt E-5, all asphalts were well above the proposed requirements for their respective grades. On the whole, these results are similar to the results obtained on penetration-graded asphalts (*4, 5*), thus indicating that viscosity grading did not materially affect ductility characteristics as measured by present specified standards for ductility at 77° F.

Inspection of the ductility results at temperatures of 45° F. for the AC-5 grade asphalts and at 45° and 60° F. for the other grades shows large differences for the asphalts from different sources. To explore what relations might exist between the viscosity data and ductility, several correlations were tried at 45° and 60° F. A good relation between ductility and viscosity existed among asphalts from the same source but none among asphalts from different sources. However, good correlations existed between ductility and shear susceptibility at 45° and 60° F. for all the asphalts tested, regardless of source. The equation derived by the method of least squares for the relation shown in figure 11 between log ductility and shear susceptibility at 60° F. is:

$$\log \text{ductility} = 3.19 - 4.88 \text{ shear susceptibility}$$

The shear susceptibility values were calculated from the tangent drawn between 0.05

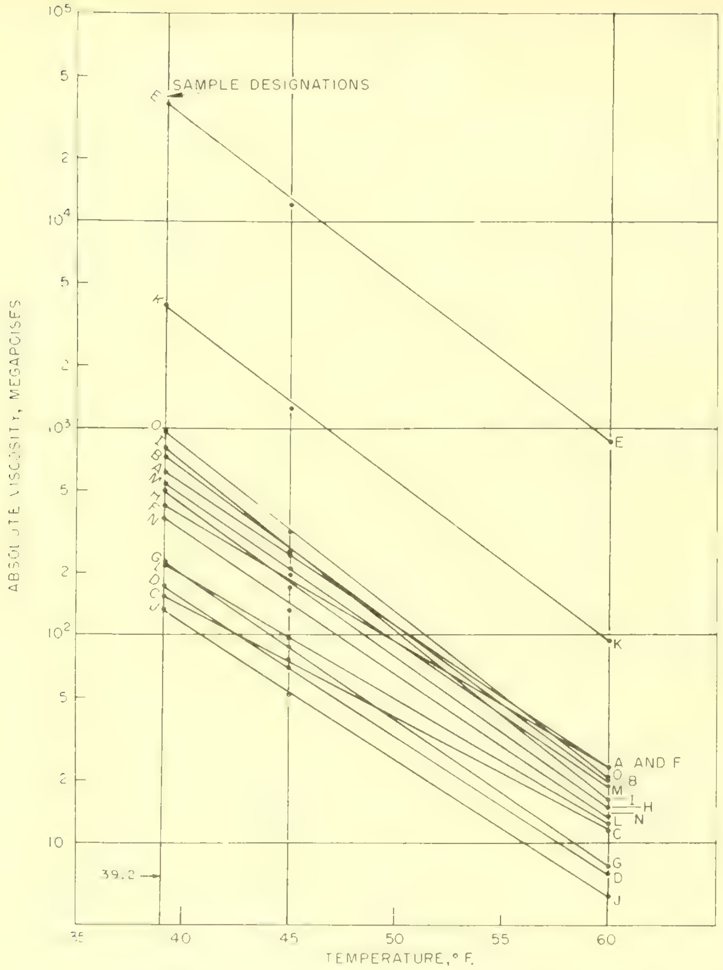


Figure 5.—Limiting viscosity of AC–10 viscosity graded asphalt cements at low temperatures.

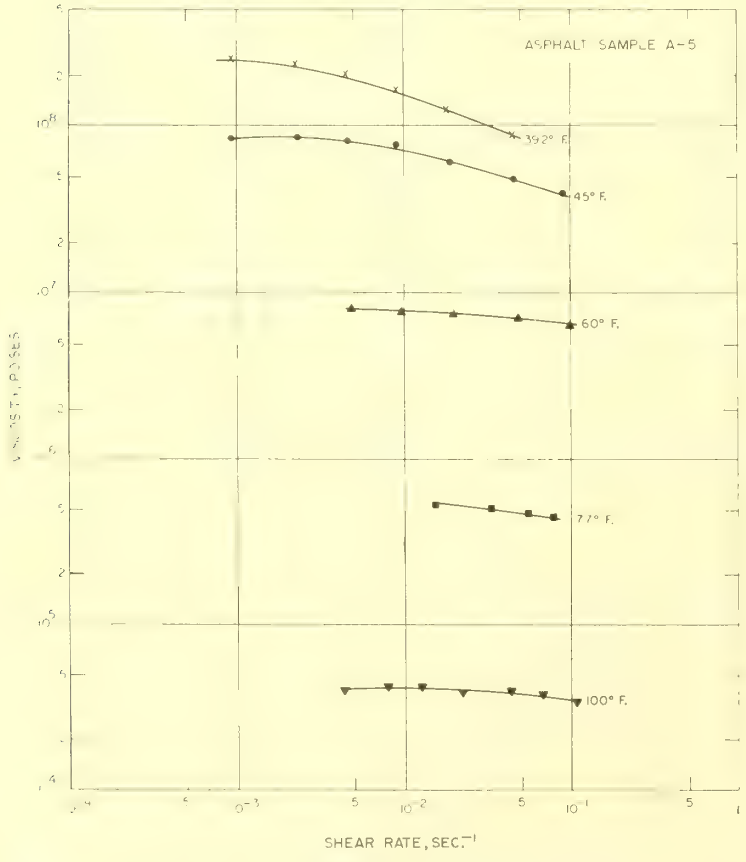


Figure 6.—Flow diagrams for asphalt A–5.

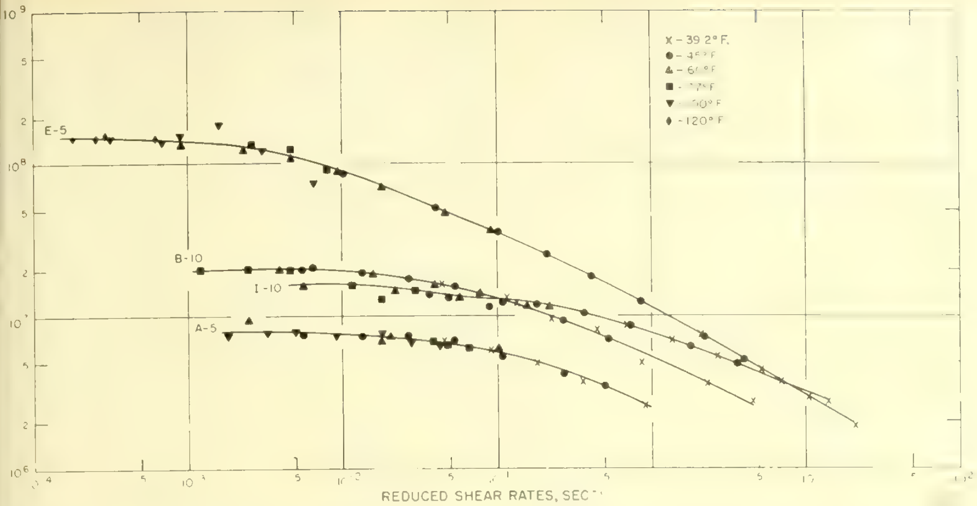
sec.⁻¹ and 0.10 sec.⁻¹ rates of shear on the viscosity-rate of shear curves obtained in the viscosity determinations. Except for asphalt I-20, those asphalts having a ductility of more than 150 cm. had shear susceptibilities less than 0.25. It is evident that this relation between ductility and shear susceptibility could provide a basis for using shear susceptibility rather than the present ductility requirements and possibly could effect an economy in testing in viscosity-graded specifications. The low temperature viscosity determination of shear susceptibility involving a simple calculation could be obtained in the viscosity test.

Durability

Starting about 1955, many specifications for asphalt cements incorporated requirements for the thin-film oven test to provide more assurance of durability, particularly resistance to hardening during hot mixing and in service. The thin-film oven test was proposed in the study specifications but a viscosity ratio based on viscosities at 140° F. before and after heating was used in place of the percent of retained penetration in the penetration grade specifications. The results of the thin-film tests on the viscosity-graded asphalts are given in table 5. To provide additional information for comparative purposes the tests made on the residues covered considerably more than was required in the study specifications.

The viscosity ratios at 140° F. were well under 5, the ratio proposed. Actually, all ratios, except that shown for asphalt P-20, were less than 4. Also, all the asphalts were within the maxima specified for percent retained penetration in both the present ASTM and AASHO specifications for asphalt cements. This would indicate that, for the asphalts studied, the proposed maximum viscosity ratio at 140° F. was either unnecessarily liberal or that borderline materials were not included in the tests. The viscosity ratios for asphalts of different grades from the same source did not differ significantly. This is in contrast to experience with penetration grade asphalts in which the percent retained penetration usually decreases with an increase in penetration.

Because the thin-film test had been developed and used on the basis of penetrations at 77° F., and the study proposal used viscosities at 140° F. as criteria, a determination was made as to whether the proposed criteria were satisfactory or whether viscosities at different temperatures would provide a better evaluation. Accordingly, viscosity ratios at both 140° F. and 60° F. were plotted against the percent of retained penetration at 77° F. for each grade, as shown for the AC-20 asphalts in figure 12. The 60° F. viscosity ratios are given at three different shear rates because at this temperature shear susceptibility could affect the results. The figure shows that good correlations were not obtained. This result is in contrast to the correlation obtained between viscosity ratio



igure 7.—*Composite master curves for reduced shear and viscosity at reference temperature* (T_o) *of 60° F. (288.7 Kelvin) for selected asphalt cements.*

nd retained penetration, both determined at 7° F., for a selected group of 85–100 pene-ation grade asphalts (*14*). A control aced at some temperature, such as 60° or 7° F. would provide a better correlation. his is another indication of the need for a low mperature control.

When the thin-film test was first proposed, ductility requirement for the residue was recommended because some asphalts showed an abnormally large loss in ductility during heating. As indicated previously, there is evidence that low ductility, caused by hardening occurring in hot mixing or in service, is associated with poor performance. Accordingly, Public Roads, AASHO, and many State specifications include a requirement for minimum ductility on the thin-film residue. However, the present ASTM specification and the proposed study specification do not include this requirement. The ductility characteristics of the thin-film residues of the asphalts are given in table 5.

Using the penetration of the original asphalt as a basis for comparison, the results at 77° F. show that the asphalts met the AASHO requirements for ductility of thin-film residue for the respective penetration grades. There is generally a substantial decrease in ductility at the 60° F. test temperature, and a specification requirement at this or a lower temperature might provide better criteria. No correlation was present between ductility and viscosity at 60° F. However, a relatively good correlation between ductility and shear susceptibility at 60° F. was obtained for all asphalts, as shown in figure 13. The equation derived by the method of least squares for the relation between log ductility and shear susceptibility of the thin-film residue at 60° F. is:

$$\text{log ductility} = 2.66 - 4.19 \text{ shear susceptibility}$$

This supports the previous finding on the original asphalts that shear susceptibility may be useful as a specification requirement in place of the ductility test. It is pointed out that each value for shear susceptibility and ductility used in this correlation was obtained by single determination.

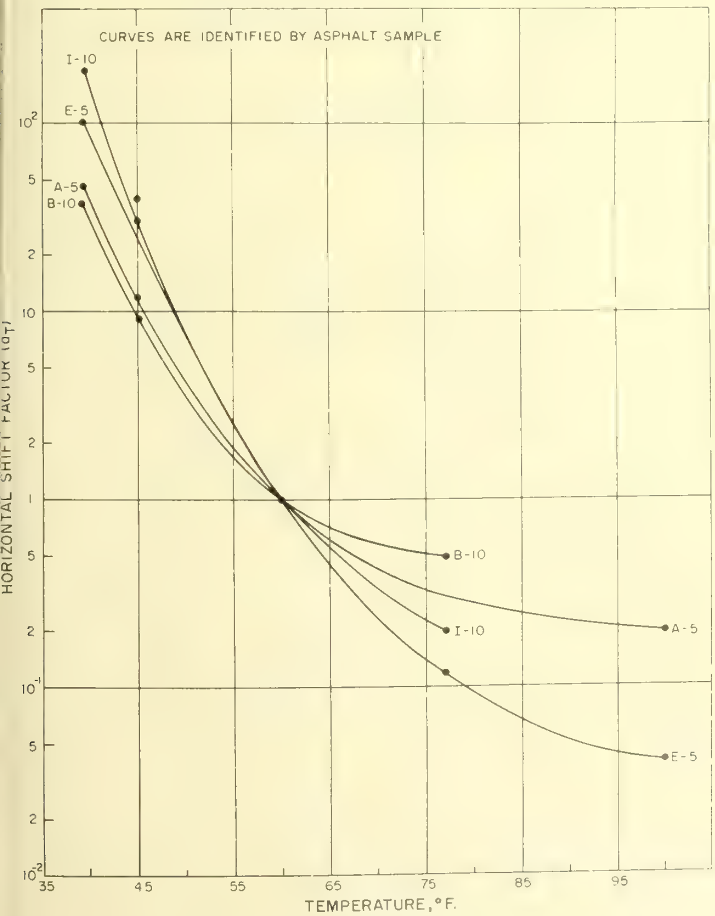


Figure 8.—*Horizontal shift factors* (a_T) *for selected asphalt cements.*

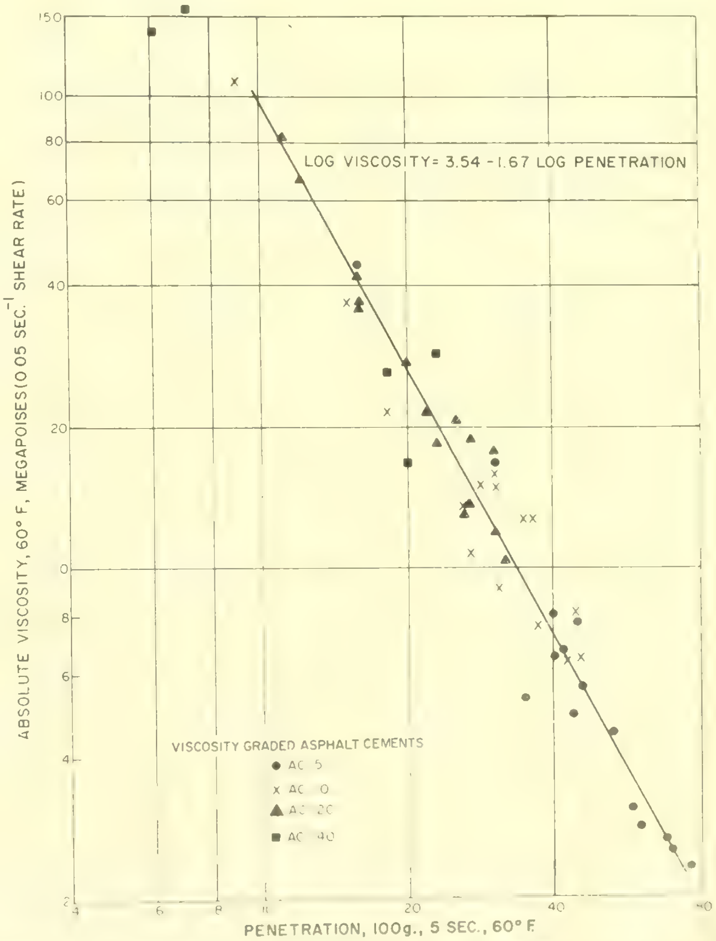


Figure 9.—Relation between penetration at 60° F. and absolute viscosity at 60° F. (0.05 sec.⁻¹ shear rate).

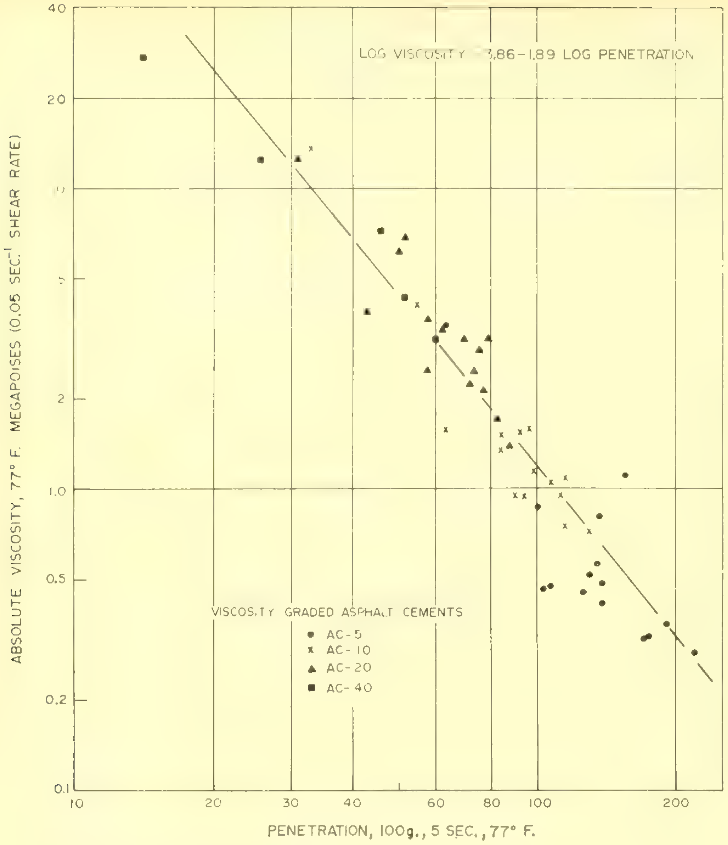


Figure 10.—Relation between penetration at 77° F. and viscosity at 77° F. (0.05 sec.⁻¹ shear rate).

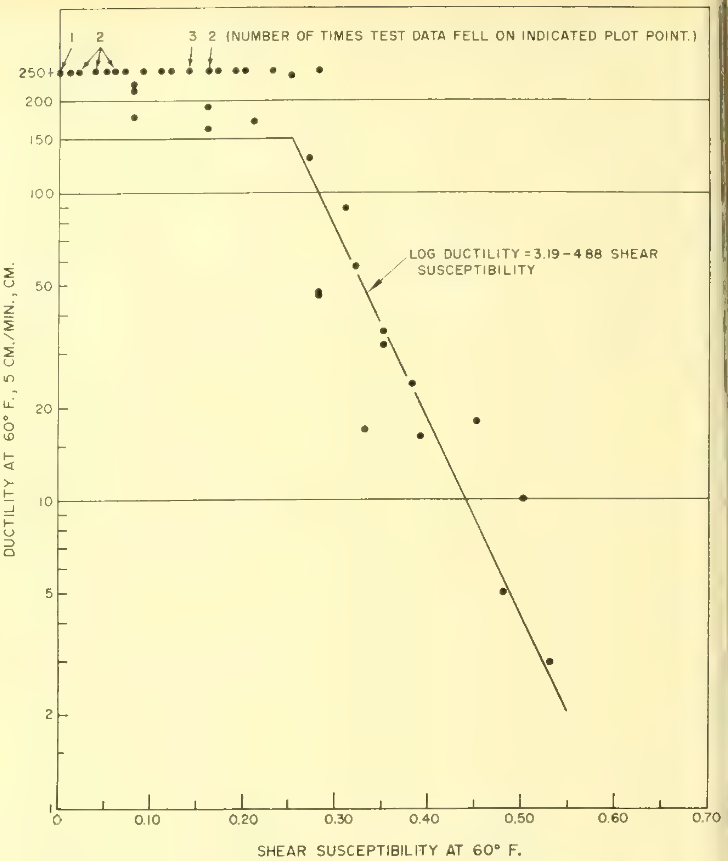


Figure 11.—Relation of ductility and shear susceptibility at 60° F. of all asphalts tested.

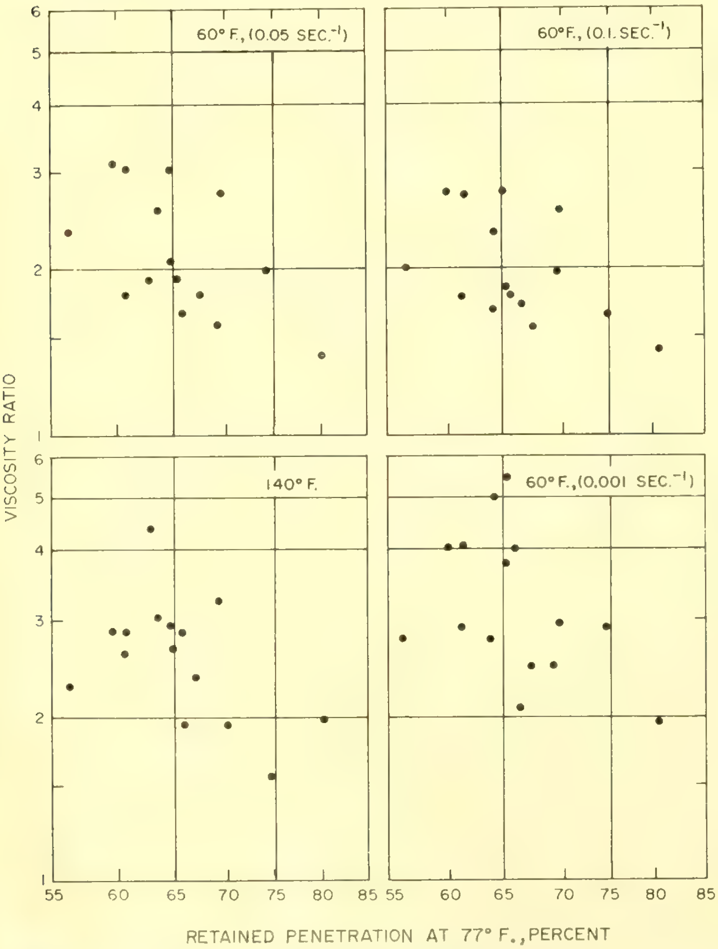


Figure 12.—Relation of percent retained peuetration at 77° F. and viscosity ratios at 140° F. and 60° F. for AC-20 viscosity grade asphalt cement.

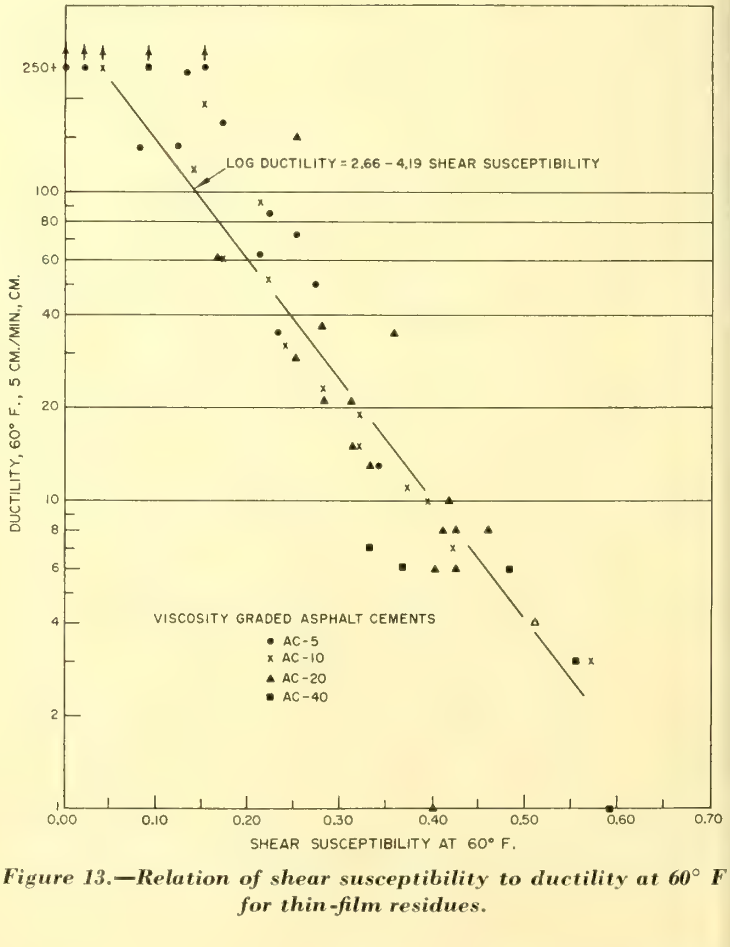


Figure 13.—Relation of shear susceptibility to ductility at 60° F for thin-film residues.

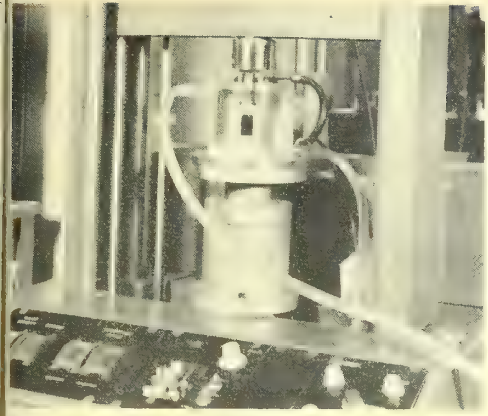
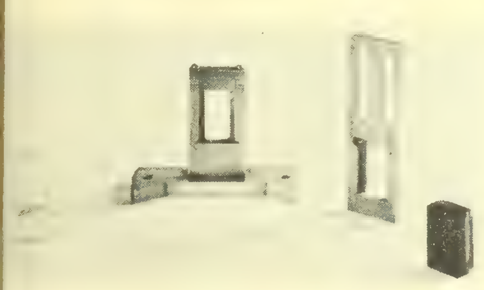
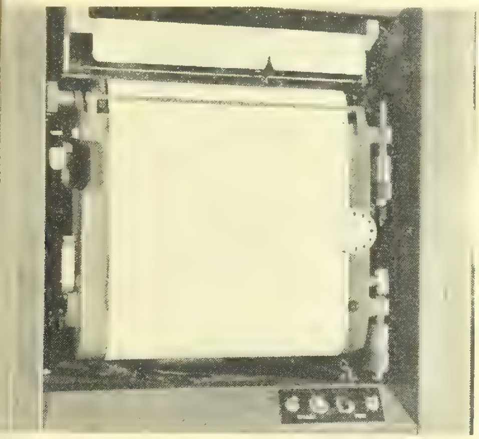


Figure 14.—Viscometer assembly during test run.



'igure 15.—Viscometer clamps, matched lass plates, and prepared test specimen.



'igure 16.—Recorder, showing load trace.

Table 8.—Viscosity results at low temperatures for selected asphalts

Temperature, ° F., and asphalt code number	Rate of shear	Viscosity
	Sec.$^{-1}$	*Megapoises*
39.2:		
A-5	4.39×10^{-2}	86.8
	1.76×10^{-2}	122
	8.78×10^{-3}	162
	4.39×10^{-3}	204
	2.20×10^{-3}	234
	8.78×10^{-4}	250
E-5	1.04×10^{-1}	188
	5.21×10^{-2}	290
	2.09×10^{-2}	509
	1.04×10^{-2}	760
	5.21×10^{-3}	1,160
	2.61×10^{-3}	1,740
	1.04×10^{-3}	2,950
B-10	6.41×10^{-2}	173
	3.21×10^{-2}	230
	1.28×10^{-2}	311
	6.41×10^{-3}	516
	3.21×10^{-3}	599
	1.60×10^{-3}	819
	6.41×10^{-4}	993
I-10	9.10×10^{-2}	167
	4.55×10^{-2}	224
	1.82×10^{-2}	335
	9.10×10^{-3}	444
	4.55×10^{-3}	540
	2.28×10^{-3}	666
	9.10×10^{-4}	778
45:		
A-5	8.78×10^{-2}	38.9
	4.39×10^{-2}	45.9
	1.76×10^{-2}	59.4
	8.78×10^{-3}	78.4
	4.39×10^{-3}	81.2
	2.20×10^{-3}	81.9
	8.78×10^{-4}	95.5
E-5	1.04×10^{-1}	124
	5.21×10^{-2}	176
	2.09×10^{-2}	293
	1.04×10^{-2}	428
	5.21×10^{-3}	614
	2.61×10^{-3}	828
	1.04×10^{-3}	1,250
B-10	6.41×10^{-2}	91.4
	3.21×10^{-2}	117
	1.28×10^{-2}	159
	6.41×10^{-3}	205
	3.21×10^{-3}	231
	1.60×10^{-3}	252
	6.41×10^{-4}	241
I-10	9.10×10^{-2}	94.2
	4.55×10^{-2}	122
	1.82×10^{-2}	161
	9.10×10^{-3}	187
	4.55×10^{-3}	213
	2.28×10^{-3}	209
	9.10×10^{-4}	250
60:		
A-5	9.46×10^{-2}	6.35
	4.73×10^{-2}	6.79
	1.89×10^{-2}	7.25
	9.46×10^{-3}	7.59
	4.73×10^{-3}	7.88
E-5	9.09×10^{-2}	35.8
	4.54×10^{-2}	46.8
	1.82×10^{-2}	67.6
	9.09×10^{-3}	87.0
	4.54×10^{-3}	109
	2.27×10^{-3}	124
	9.09×10^{-4}	136
B-10	1.54×10^{-1}	11.7
	7.72×10^{-2}	14.0
	3.86×10^{-2}	15.9
	1.54×10^{-2}	18.9
	3.86×10^{-3}	19.6
I-10	2.19×10^{-1}	11.5
	1.09×10^{-1}	12.5
	5.47×10^{-2}	13.6
	2.19×10^{-2}	14.1
	5.47×10^{-3}	14.8
77:		
A-5	7.41×10^{-2}	.439
	5.32×10^{-2}	.460
	3.20×10^{-2}	.509
	1.51×10^{-2}	.536
E-5	6.60×10^{-2}	3.00
	5.40×10^{-2}	3.10
	3.10×10^{-2}	4.20
	1.75×10^{-2}	4.60
B-10	1.14×10^{-1}	.730
	2.95×10^{-2}	1.10
	1.55×10^{-2}	1.10
I-10	1.00×10^{-1}	1.15
	6.00×10^{-2}	1.35
	2.40×10^{-2}	1.35
100:		
A-5	9.79×10^{-2}	.033
	6.40×10^{-2}	.038
	4.07×10^{-2}	.040
	2.14×10^{-2}	.038
	1.17×10^{-2}	.042
	7.71×10^{-3}	.042
	4.22×10^{-3}	.039
E-5	1.58×10^{-1}	.052
	7.45×10^{-2}	.088
	3.88×10^{-2}	.126
	2.34×10^{-2}	.105
	1.66×10^{-2}	.098
	7.77×10^{-3}	.105
120:		
E-5	2.79×10^{-1}	.0058
	1.34×10^{-1}	.0061
	1.15×10^{-1}	.0057
	8.62×10^{-2}	.0057
	6.81×10^{-2}	.0048
	3.67×10^{-2}	.0044

Controlled Shear Rate Viscometer Test Method

The method for the determination of the iscosity of paving grade asphalt by use of a liding plate viscometer at controlled rates of hear is applicable to materials having visosities in the range of 10^5 to 10^{10} poises and is herefore suitable for use at temperatures f 4.0° C., 15.6° C., and so on, where viscosity s in the range indicated. Its use also is uitable for determinations on materials havng either Newtonian or non-Newtonian flow roperties. Shear susceptibility may also be etermined. The details of the method of est are given in reference (*7*).

An asphalt film of known thickness is formed etween matched pairs of aluminum or glass lates. One plate is clamped in a fixed posiion, the other is displaced at constant, reselected velocities, and the constant shearing force is measured. Rate of shear, shear stress, and viscosity are calculated from the dimensions of the specimen, displacement velocity, and the shearing force developed in the test. When several displacement velocities are used, shear susceptibilities may also be determined for non-Newtonian materials. A non-Newtonian complex liquid is a liquid in which the rate of shear is not proportional to the shearing stress. This method measures the viscous flow behavior of asphalt at relatively low shearing rates (1 to 10^{-4} sec.$^{-1}$) and low temperature ranges (15.6° C. and below).

Table 9.—Horizontal shift factors (a_T) for different temperatures

Temperature	Asphalt code number			
	A-5	E-5	B-10	I-10
° F.	a_T	a_T	a_T	a_T
39.2	4.6×10	1.0×10^2	3.7×10	1.9×10^2
45	1.2×10	3.0×10	9.0×1	4.0×10
60	1.0×1	1.0×1	1.0×1	1.0×1
77	1.4×1	1.2×10^{-1}	5.0×10^{-1}	2.0×10^{-1}
100	2.0×10^{-1}	4.0×10^{-2}		
120		4.0×10^{-3}		

A sliding plate viscometer is shown in figure 14 and the clamps, matched glass plates, and a test specimen are shown in figure 15. A load trace on the recorder is shown in figure 16.

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A Comparative Evaluation of Trip Distribution Procedures

BY THE OFFICE OF PLANNING BUREAU OF PUBLIC ROADS

Reported by[1] KEVIN E. HEANUE, Highway Engineer, Urban Planning Division, and CLYDE E. PYERS[2]

The results of a research project designed to test, evaluate, and compare four major trip distribution techniques are presented in this article. These techniques are the Fratar growth factor procedures as developed by Thomas J. Fratar and utilized for many transportation studies; the so-called gravity model currently the most widely used of the mathematical travel formulas; the intervening opportunities model developed by Morton Schneider of the Chicago Area Transportation Study (CATS) and since then utilized for several other major studies; and the competing opportunities model suggested by Anthony Tomazinis of the Penn-Jersey Transportation Study (P-J), but as yet not utilized in an operational study.

These techniques present interesting contrasts in their approach to the trip distribution problem. People as social beings do not order their lives according to strict physical or mathematical laws and no single model could ever be expected to perfectly match reality; however, some theories can be expected to be more explanatory than others. In this article an attempt has been made to give the potential user of trip distribution models insight into the theoretical differences underlying the models, as well as knowledge of some of their advantages and disadvantages in a practical application. The validity of these models as forecasting tools is also presented in an analysis of the accuracy of model forecasts made for a 7-year historical period for Washington, D.C.

Introduction

THE RAPID evolution of computer-oriented trip distribution techniques coupled with the pressing deadlines of the major urban transportation studies has made it difficult to start a comprehensive program for testing and evaluating the most widely used trip distribution techniques. Individual applications of trip distribution models often have involved a certain amount of research, and as a byproduct of these applications, revisions and improvements in each of the techniques have been made. However, since about 1963, the rate of evolutionary development has slackened to the extent that most of the techniques are now considered to be mature.

This article is a report on the results of a research project conducted by the Urban Planning Division of the Bureau of Public Roads to test, evaluate, and compare four major trip distribution techniques. These are the Fratar growth factor procedures as developed by Thomas J. Fratar and utilized for many transportation studies (*1*)[3]; the so-called gravity model, currently the most widely used of the mathematical travel formulas (*2*); the intervening opportunities model developed by Morton Schneider of the Chicago Area Transportation Study (CATS) and since then utilized for several other major studies (*3*); and the competing opportunities model suggested by Anthony Tomazinis of the Penn-Jersey Transportation Study (P-J), but as yet not utilized in an operational study (*4*).

The mathematical model techniques differ in the approach to the trip distribution problem. These models can be classified into two categories: growth factor procedures and interarea travel formulas. For the growth factor procedures, growth factors that reflect land-use changes in the zones are used to expand a known travel pattern to some future year. The interarea travel formulas simulate travel distributions by relating them to characteristics of the land-use pattern and of the transportation system. The interarea travel formulas require calibration; that is, a determination of the effect of spatial separation on travel, prior to their actual application as forecasting tools.

Conclusions

On the basis of the research reported in this article, it is concluded that the gravity model and the intervening opportunities model had about equal reliability and utility in simulating the 1948 and 1955 trip distributions for Washington, D.C. Although use of the Fratar growth factor procedure correctly expanded trips for stable areas, this procedure had significant weaknesses when applied to areas undergoing land-use changes.

The competing opportunities model, in exploratory work in the Penn-Jersey study at a district level (grouping of zones), offered promise as a useful tool. However, it is concluded that this procedure may not be useful for determining trip distributions between traffic zones as small as the Washington, D.C. zones.

Study Procedures

For the study reported here, an attempt was made to establish a standard set of test conditions for evaluating the four procedures. It was not possible always to adhere to strictly comparable conditions but each variation from a common base is fully discussed.

Basic data sources for the analysis were the 1948 and 1955 home interview travel surveys conducted in Washington, D.C. The 1948 survey covered 5 percent of the dwelling units in the metropolitan area. In 1955 a repeat survey was conducted. In the repeat survey, occupants of 3 percent of the dwelling units were interviewed within the District of Columbia. Elsewhere in the area, occupants of 10 percent of the dwelling units were interviewed. Figure 1 is a map of the study area.

The boundaries of the 1948 and the 1955 study areas were not exactly matched. Every attempt was made, however, to make the 1948 and 1955 analysis zones compatible. This was not a critical problem when the interarea travel formulas were used, as the only variable projected directly is the effect of spatial separation on trip making. This variable is independent of zone configuration. The Fratar procedure, however, requires compatible zones for base and projection years. For the Fratar analysis it was necessary to reduce the 400 zones used in the standard analysis to 362 units that were comparable. Mostly, this involved eliminating zones that were external to the 1948 study area but internal to the 1955 study area and thus had zero trip ends in 1948. Certain irregularities in zonal boundaries still were present; however, their effect was not serious. Because of changes in the location of external cordon stations between 1948 and 1955, all trips crossing the cordon (external trips) were omitted from the analysis. The basic trips considered were the total person trips by all modes expanded from the home interview surveys. Trips recorded in the special truck and taxi surveys were not included.

Although the test period covered by this analysis was only 7 years, the characteristics of the area changed significantly in this time.

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[1] Presented at the 44th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1965.

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[3] Numbers in parentheses identify references listed on page 51.

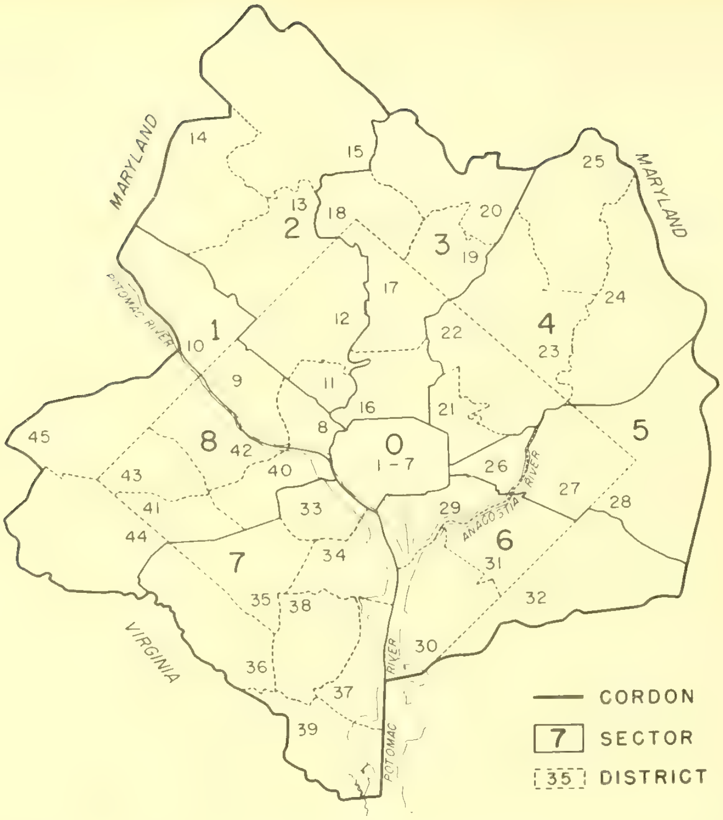

Figure 1.—Study area for O-D surveys, Washington, D.C., 1948 and 1955.

The population increased 38 percent to almost 1.5 million; the number of person trips increased by more than 42 percent; and the number of passenger cars owned by residents increased 96 percent, almost double. Probably the most significant change in the study area, during the 7-year period, was the decentralization of many activities. Residential, employment, and shopping activities all were relatively less oriented to the central business district (CBD) in 1955 than in 1948 (*5*). Total trips to the CBD decreased relatively from 28 percent to 21 percent of the total person trips.

The study reported here was designed so that the 1948 survey data would be used as the base year travel pattern for the Fratar procedure and as a calibration source for the interarea travel formulas. The 1955 travel survey data were used as a control against which all forecasts were checked. To establish the Fratar growth factors, trip ends reflecting the 1955 characteristics were taken directly from the 1955 O-D survey trip ends. Also, these trip ends were used directly as producing and attracting powers of the zones when the synthetic distributions were calculated with the interarea travel formulas. The 1955 trip ends were used rather than estimates developed in a land use and trip generation analysis, to restrict the possible sources of error to those inherent within each of the distribution procedures.

Fratar Procedure

The Fratar procedure has been proved to be computationally the most efficient of the growth factor techniques (*6*). The basic premise of the Fratar procedure is that, the distribution of trips from a zone is proportional to the present movements out of the zone modified by the growth factor of the zone to which the trips are attracted. The future volume of trips out of a zone is determined from the present trips out of the zone and the growth factors developed for the zone. In previous applications of the Fratar procedure, generally only one trip purpose had been used. The Urban Planning Division of the Bureau of Public Roads, in 1962, developed a Fratar procedure that uses up to 10 trip purposes. This program also permits the application of growth factors by mode, time of day, or for trips entering or leaving a zone. The basic formula for the directional purpose Fratar procedure is:

$$T_{ij} = t_{ij}G_iG_j\left(\frac{L_i + L_j}{2}\right)$$

Where,

T_{ij} = Future year trips from zone i to zone j with a given purpose at zone i and a given purpose at zone j.

t_{ij} = Base year trips between zone i and zone j with a given purpose at zone i and a given purpose at zone j.

G_i = Growth factor for zone i for a given purpose.

L_i = Locational factor

$$= \frac{t_i}{\sum_{j=1}^{n} t_{ij}F_j}$$

t_i = Base year trip ends at zone i for a given purpose.

The directional purpose Fratar allows th procedure to be sensitive to the type of lan use changes that are occurring in a given zon For example, work trips can be expanded as function of employment changes only. Prio to the development of the new compute program, all trips regardless of purpose wer expanded by a measure of the overall growt of the zone.

Gravity Model

The gravity model is the most thoroughl documented of the trip distribution technique (*7*, *8*, *9*, and *10*). This technique, loosel paralleling Newton's gravitational law, i based upon the assumption that all trip starting from a given zone are attracted b the different traffic generators and that thi attraction is in direct proportion to the siz of the generator and in inverse proportion t the spatial separation between the areas The Public Roads computer battery gravit model program was used in the research reported here. The basic gravity model formulation is:

$$T_{ij} = \frac{P_iA_jF_{ij}K_{ij}}{\sum_{j=1}^{n} A_jF_{ij}K_{ij}}$$

Where,

T_{ij}= Trips produced in zone i and attracted to zone j.

P_i= Trips produced in zone i.

A_j= Trips attracted to zone j.

F_{ij}= Empirically derived traveltime factors (one factor for each 1-minute increment of traveltime) that are a function of the spatial separation between the zones. These factors express the average areawide effect of spatial separation on trip interchange

K_{ij}= Specific zone-to-zone adjustment factor to allow for the incorporation o the effect on travel patterns of defined social or economic linkages no otherwise accounted for in the gravity model formulation.

The traveltime factors are developed in an iterative procedure that is continued until the synthetic trips calculated for each trip-length interval closely match the surveyed trips reported for the same intervals. Any convenient set of traveltime factors may be used to start the iteration procedure.

Intervening Opportunities Model

For the intervening opportunities model, a probability concept is used that in essence requires a trip to remain as short as possible and to be lengthened only when a prior destination is not acceptable. An equal areawide probability of acceptance for any origin is defined for all destinations in a given category. In use of this model, all trip destinations (opportunities) are considered in sequence by traveltime from zone of origin. The first destination considered is the one losest to the origin of the trip and its acceptnce has the stated areawide probability of cceptance. The same basic probability of cceptance exists for the next opportunity but he actual probability of its being accepted s decreased by the possibility of the tripnaker having accepted the first opportunity s his destination. The procedure is applied o each successive destination from point of rigin but, for each successive opportunity, he actual probability of its being accepted ecreases. Thus, spatial separation for the ntervening opportunities model is measured n terms of the number of intervening destiations rather than in terms of the absolute raveltime, cost, or distance between one one and the other. The intervening opporunities are determined by arraying the vailable destinations in all zones by traveltime rom the zone of origin. The formulation or the procedure is:

$$T_{ij}=O_i[e^{-LD}-e^{-L(D+D_j)}]$$

Vhere,

T_{ij}= Trips originating in zone i with destinations in zone j.

O_i= Trip origins in zone i.

D= Trip destinations considered prior to zone j.

D_j= Trip destinations in zone j.

L= Measure of probability that a random destination will satisfy the needs of a particular trip. It is an empirically derived function that describes the rate of trip decay with increasing trip destinations and increasing length of trip.

e= Base of natural logarithms (2.71828).

This model is calibrated by varying the probability values until the simulated trip distribution reproduces the person hours of travel and percent intrazonal trips of the surveyed trip distribution.

Competing Opportunities Model

Essentially the basic concept of the competing opportunities model is that opportunities or destinations compete for trips within equal traveltime, travel distance, or travel cost bands as measured from the zone of origin. Within a given band, each destination has an equal probability of acceptance. The probability that trips will be distributed to a certain zone is the product of two independent probabilities. The first, called the probability of satisfaction, reflects the chances that a trip will be of a particular length and is a function of the destinations at a greater distance than the time band under consideration. The determination of the specific destination within this time band is quantified by a probability of attraction, which is related to the available opportunities that fall within the area up to and including the time band considered. The mathematical formulation for this procedure is:

$$T_{ij}=O_i p_{a_j} p_{s_j}$$

Where,

T_{ij}= Trips produced in zone i and attracted to zone j.

O_i= Trip origins in zone i.

p_{a_j}= Probability of attraction

= destination available in zone j divided by sum of destinations available in time bands up to and including band m.

$$=\frac{D_j}{\sum_{k=0}^{m} D_k}$$

p_{s_j}= Probability of satisfaction

= 1 minus the sum of the destinations available in time bands up to and including band m divided by the sum of total destinations in study area.

$$=1-\frac{\sum_{k=0}^{m} D_k}{\sum_{k=0}^{n} D_k}$$

k= Any time band.

m= Time band into which zone j falls.

D_k= Destinations available in time band k.

n= Last time band as measured from origin zone i.

D_j= Destinations available in zone j.

This model is calibrated by varying the width of the attracting bands until the trip length characteristics of the synthetic trips correspond to the trip length characteristics of the surveyed trips.

Basic Tests

Four basic tests were used to measure the ability of the different procedures to reproduce the total person trip movements of the known travel patterns. These tests evaluated the procedures as to: (1) ability to match the trip length frequency distribution from the O–D survey; (2) ability to produce river crossing volumes that matched O–D survey volumes; (3) ability to match O–D survey trip movements by corridor to and from the CBD, and (4) accuracy of model as measured by statistical comparison of O–D survey trips and model trips assigned to a spider network.

The Fratar procedure could not be tested against 1948 data because its base is the survey data. However, some validation for the other travel formulas was accomplished against base conditions. Such validation is an essential part of calibrating the models before they are used for projection. The accuracy of this base year simulation is typically the most important check in the calibration procedure. This check is based on the fact that the calibrated travel model must accurately simulate the base year travel pattern before the same model can be expected to simulate accurately a travel pattern for a future year.

The trip length frequency comparisons were made by 1-minute time intervals. A comparison of the O–D and model trip length frequency curves and mean trip lengths provides a measure of the accuracy of the estimate of person hours of travel for the total area. Such a comparison also provides an indication of the accuracy of the trip distribution.

The river crossing tests were made on the basis of screenlines set up on the Potomac and Anacostia Rivers. Because of the trip definition, the base screenline values were the O–D survey person movements rather than actual vehicle counts. The analysis of movements by corridor to and from the CBD was designed to detect any bias in the estimated travel patterns. The gravity model computer program provides for the use of adjustment factors to correct for bias. When the other techniques are used, it is usually assumed that the procedure adequately distributes trips and that adjustment factors are not needed.

The statistical analysis of trips assigned to a spider network was used as the fourth test on the procedures. A spider network consists of airline distance connections between adjacent zone centroids. The resultant differences between the O–D and model assignments were arrayed by volume group and the root mean square error was calculated. This test provides a measure of the overall accuracy of the final trip distribution.

Calibration of Interarea Travel Formulas

Gravity model

In previous research with the gravity model, the Washington, D.C., 1955 O–D data were used as a calibration base rather than the 1948 data (*7*, *8*). The model parameters were in effect forecast backward from 1955 to 1948. For the research reported here, the gravity model was recalibrated and the 1948 O–D data were used as a base. These 1948 model parameters were used to forecast 1955 travel patterns. The research results showed that the same traveltime factors held for both 1948 and 1955 and that the K factors (socioeconomic adjustment factors) also had the same relation with average family income by district for both periods. Some doubt existed as to whether the river crossing time impedances could have been properly forecast without the knowledge gained from the former research, but for comparisons made in the study reported here it was assumed that the river barriers could be properly forecast. The impedances varied from 5 and 3 minutes for work and nonwork trips in 1948, to 6 and 5 minutes in 1955, respectively. The 1955 river crossings were forecast from 1948 on the basis of the relative congestion levels for the 2 years (*8*, p. 26). The traveltime factors for each of the six trip purposes used for both 1948 and 1955 are shown in table 1.

Intervening opportunities model

Several methods of calibration of the intervening opportunities model were tried for the 1948 Washington area data. The best procedures and the final calibration parameters were incorporated into the study discussed here. The several methods of calibration and the resultant findings are documented in

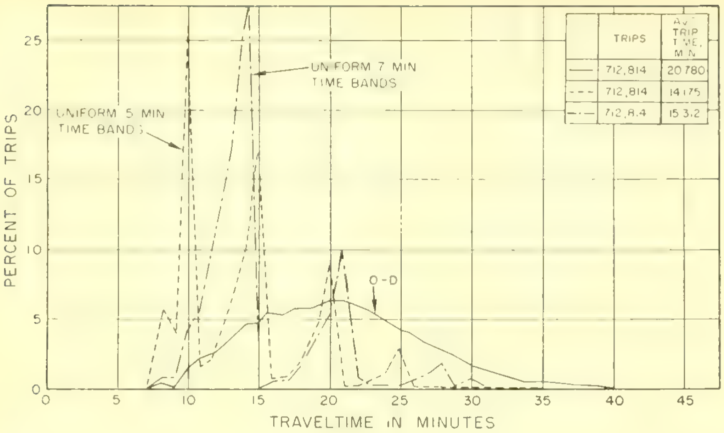

Figure 2.—Work trip length distribution in uniform time bands from O–D survey and competing opportunities model, Washington, D.C., 1948.

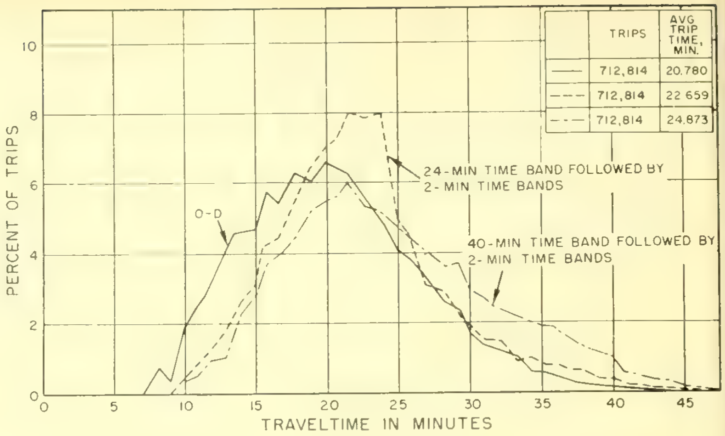

Figure 3.—Work trip length distribution in different time bands from O–D survey and competing opportunities model, Washington, D.C., 1948.

reference (*11*). The methods of calibration and forecasting of the model examined are very close to those used previously in Chicago and elsewhere except that procedures were developed to ensure that the model would both send and attract approximately the correct number of trips for each zone in the study area. Without these adjustments, only 84 percent of the total trips were distributed and trips to the CBD were overestimated by 20 percent.

Trip ends were stratified into long residential, long nonresidential, and short. Both long and short *L* values were developed through an iterative process to ensure that, when the final *L* values were applied to the appropriate trip ends, a satisfactory average trip length, trip length frequency curve, and number of intrazonal trips would be obtained for the total trips—all three trip types combined.

The river crossing time impedance was needed for the intervening opportunities model as well as for the gravity model. The additional bridge crossing time required for the 1948 intervening opportunities model calibration was 5 minutes. The use of procedures developed in the gravity model research to forecast the impedance for the intervening opportunities model estimated that an 8-minute impedance was required for 1955 data. Although use of this 8-minute forecast time penalty did materially improve model accuracy, estimated Potomac River crossings by the intervening opportunities model were approximately 16 percent high. The differences in forecast time penalty by the gravity model and the intervening opportunities model were caused by the different trip purpose categories, which required different weighting of peak hour trips. The basic structure of the models also made it necessary to use different impedances for 1948 data.

An increase in the total number of trip destinations or opportunities requires that the probability of any one of these destinations being accepted for any given origin be reduced. Because of the growth in total and intrazonal trips in the study area, the 1948 *L* (probability) values required reduction for use in 1955 forecasts. The final 1948 long and short *L* values were 2.50×10^{-6} and 13.00×10^{-6}, respectively, and they were reduced to 1.65×10^{-6} and 10.80×10^{-6} for the 1955 forecasts. These adjustments were made on the basis of the actual growth in total destinations between 1948 and 1955 (*11*).

Competing opportunities model

The competing opportunities model proved to be very difficult to calibrate. Because no systematic calibration procedures were available, many alternate approaches were tried. Initially, equal time bands were used for work trips but this was not successful, as shown in figure 2. Time bands of different widths were utilized and the results became more meaningful. It seemed that the best simulation for work trips was obtained when the first time band incorporated the majority of the opportunities in the study area. This broad band was followed by equal 2-minute bands. Even with this approach, however, it was impossible to obtain a trip length frequency distribution that approached the O–D trip length frequency. As shown in figure 3, for a 24-minute time band the peaks of the curve are too high and the curve for the 40-minute time band, which is similar in shape to the O–D curve, is offset approximately 4 minutes to the right. No grouping of time bands that would fit the O–D curve could be determined. The calibration attempts were stopped at this point.

The calibration of the competing opportunities model in the Penn–Jersey (*4*) area involved a district rather than zonal analysis. This in effect restructured the grouping of opportunities by greatly increasing the number of intrazonal trips. To date a calibration at the zonal level has not been attempted in the Penn–Jersey study. For purposes of the research reported here, the authors believe that the model would have to prove operational at the zonal level to be of universal value. District analysis was not attempted as a part of the subject research. The only other difference from the Penn–Jersey application involved the measure of spatial separation. Because of the grossness of the measure, particularly of the first opportunity band, where all trips in a ±20-minute time band would be treated equally, the use of

Table 1.—Traveltime factors by trip purpose, Washington, D.C., 1948 and 1955

Traveltime, minutes	Traveltime factors for					
	Home based trips, by purpose					Nonhome based trips
	Work	Shopping	Social and recreational	School	Miscellaneous	
1	1,000	8,700	2,000	4,200	2,600	1,600
2	1,000	8,700	2,000	4,200	2,600	1,600
3	1,000	8,700	2,000	4,200	2,600	1,600
4	1,000	8,700	2,000	4,200	2,600	1,600
5	1,000	8,700	2,000	4,200	2,600	1,600
6	1,000	8,700	2,000	4,200	2,600	1,600
7	1,000	8,700	2,000	4,200	2,600	1,600
8	1,000	8,700	2,000	4,200	2,600	1,600
9	680	5,400	1,475	2,800	1,700	1,100
10	500	3,600	1,100	2,000	1,200	780
11	400	2,300	820	1,475	875	580
12	320	1,600	640	1,075	650	440
13	270	1,120	500	800	500	340
14	235	800	400	625	390	265
15	205	580	320	480	300	215
16	180	420	260	370	235	170
17	160	310	220	280	190	140
18	145	235	180	215	150	110
19	130	180	152	165	125	92
20	120	140	130	135	105	78
21	110	105	110	110	87	65
22	100	95	95	90	72	54
23	93	70	82	70	60	46
24	87	58	72	57	51	40
25	82	45	64	47	43	33
26	77	38	56	40	38	29
27	70	32	49	32	32	25
28	63	26	42	26	28	22
29	58	21	38	22	24	20
30	53	17	34	18	21	17
31	49	13	30	15	18	15
32	44	10	27	12	15	13
33	40	8	24	10	13	12
34	37	6	21	9	12	10
35	34	5	19	7	10	9
36	29	4	17	6	8	8
37	27	3	15	5	7	6
38	24	2	13	4	6	5
39	22	2	11	4	5	4
40	19	1	10	3	4	3
41	17	0	8	3	3	2
42	15	0	7	2	3	1
43	13	0	6	2	2	0
44	11	0	5	2	1	0
45	9	0	4	1	1	0
46	7	0	3	1	0	0
47	6	0	3	1	0	0
48	5	0	2	1	0	0
49	4	0	1	0	0	0
50	3	0	1	0	0	0
51	3	0	0	0	0	0
52	2	0	0	0	0	0
53	2	0	0	0	0	0
54	1	0	0	0	0	0
55	1	0	0	0	0	0

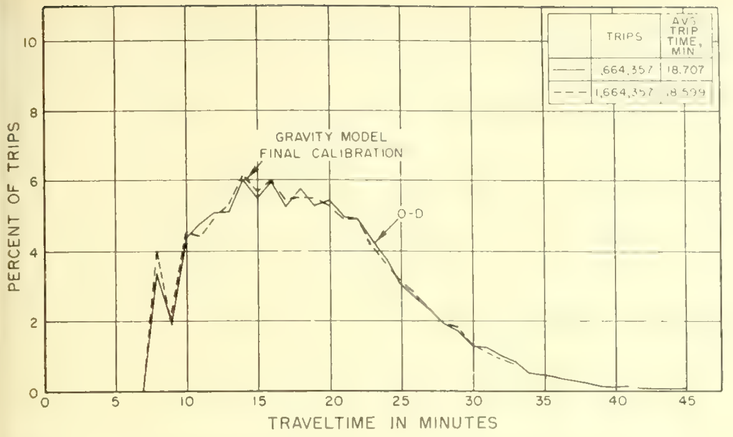

Figure 4.—Trip length distribution for total trips for all purposes from O-D survey and final calibration of gravity model, Washington, D.C., 1948.

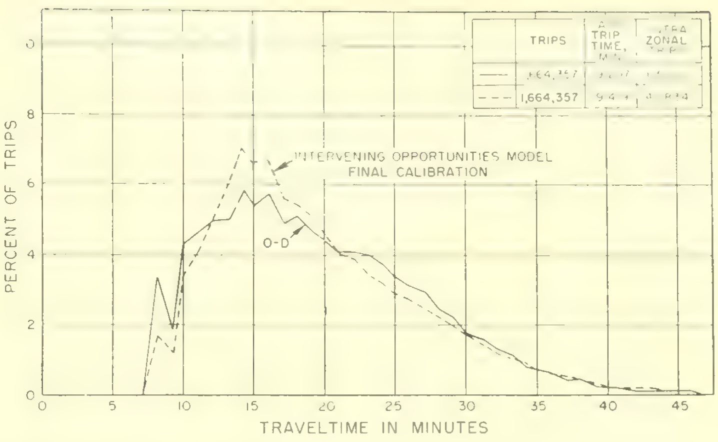

Figure 5.—Trip length distribution for total trips for all purposes from O-D survey and final calibration of intervening opportunities model, Washington, D.C., 1948.

traveltime rather than travel costs as the measure of spatial separation seems justified.

Analytical Tests

The analytical tests considered as a group show the measures of accuracy of the different procedures as well as providing data that permit insight into the theoretical differences underlying the techniques. Some of the questions that can be considered are: (1) Do urban residents maintain a continuum of travel patterns over time modified only by the growth of the area as reflected in the Fratar procedure; (2) or, when considering making a trip, do residents follow gravitational concepts weighing all attractors in direct proportion to the size of the attractors and in inverse proportion to the spatial separation as measured by the traveltime between the zones; (3) or can travel patterns be best explained by opportunity concepts in the intervening opportunities model that assumes people do not consider time directly but rather consider opportunities in sequence by traveltime and proceed on to any specific opportunity only after having considered and rejected all closer opportunities; (4) or does a person consider all opportunities in rather broad time or cost bands with all opportunities in a given band having an equal probability of acceptance as in the competing opportunities model.

People as social beings do not order their lives according to strict physical or mathematical laws and no single model could ever be expected to perfectly match reality; however, some theories can be expected to be more explanatory than others. The tests and results of use of the different models should then be analyzed as to whether (1) the particular procedure is rational; (2) the application is simple enough so that the procedure may be applied in urban planning studies by those who lack detailed experience in the procedure gained by research or earlier applications, (3) the specific procedure fits the urban area to be studied, for example, where there are local conditions such as relatively slow or rapid growth, inherent socio-economic trip linkages, large analysis units, that might make one or more of the procedures more applicable. Some underlying differences in the procedures are described in the following paragraphs. One of the most relevant differences is the weight placed on the role of traveltime as an influence on trip distribution.

In use of the Fratar growth factors procedures, the existing travel patterns are expanded by considering growth in each portion of the study area without any specific consideration of the transportation network. Should changes in the traveltime between zones be sufficient to cause a change in travel patterns in the forecast year, the Fratar or any other growth factor technique would not reflect the change in travel patterns. However, each of the interarea travel formulas studied—gravity, intervening opportunities, and competing opportunities uses time separation as a key variable. Thus changes in the transportation system and the concomitant changes in accessibility between certain portions of the study area are directly reflected in the models.

The gravity model uses a traveltime factor for each 1-minute increment and, therefore, makes the most explicit use of absolute traveltime of any of the procedures. These traveltime factors are adjusted in the calibration process until there is close agreement between the estimated trip length frequency curve and the actual curve at all increments of traveltime. These factors, or relative weights of making trips of certain lengths, are then usually assumed to remain constant over the forecast period.

In contrast to the gravity model, the intervening opportunities model does not make such explicit use of absolute traveltime. Traveltime is used instead to rank all possible destination zones from a particular origin zone. This ranking then is used to determine the number of intervening opportunities; that is, the number of destinations already considered before a particular destination zone is considered. Changes in the transportation system and accessibility between zones over the forecast period are thus reflected in the forecasting model. Two probability factors generally described as the long and short L values are used in conjunction with the intervening opportunities model to determine trip interchanges between zone pairs. The procedure of ranking used in the intervening opportunities model does cause situations unique to this model. For example, those traveling from zones in the developed area and not finding a suitable destination would ignore time gaps and immediately consider opportunities in a fringe community. In effect, any separation, or gap, would be ignored in use of the intervening opportunities model. But, when the gravity model is used, these separations are considered and the possibility of a trip crossing a gap is decreased.

The competing opportunities model is somewhat unique: results from its use approach those of the gravity model if small time bands are used but when large time bands are used, the model tends to ignore spatial separation.

In evaluating and comparing the results of the tests, consideration should be given to the formulation and parameter makeup of each of the procedures. The amount of the actual O-D data used for the base calibration and the number of parameters requiring forecasting are important in weighing the results of one model against results of others. For the Fratar procedure, all of the base year travel data from the home interview survey were used. The travel models all required less O-D data than the Fratar procedure; however, the amount of data used, as well as the number of parameters used to represent these data, varied to a considerable degree between the travel models tested.

Trip Length Frequency

Base year

Comparisons of the final calibrated model trip length frequency curves with actual trip length frequency curves: for the gravity model, the intervening opportunities model, and the competing opportunities model can be made from data shown in figures 4–6. Each of these plots is on a slightly different basis because of the way in which the research was carried out. However, each is compatible with the survey data with which it is shown.

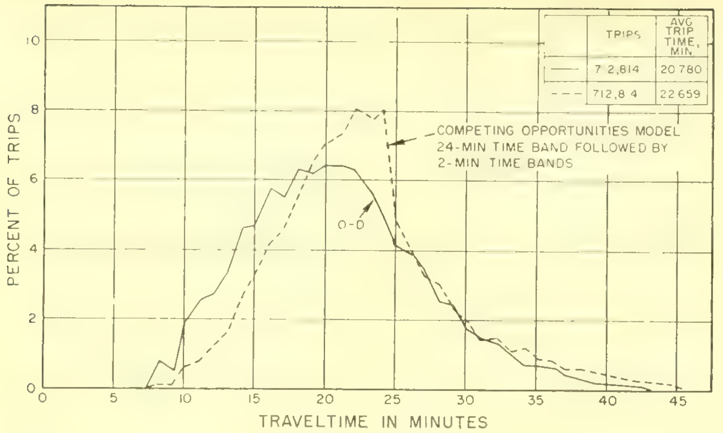

Figure 6.—Work trip length distribution from O-D survey and best calibration of competing opportunities model, Washington, D.C., 1948.

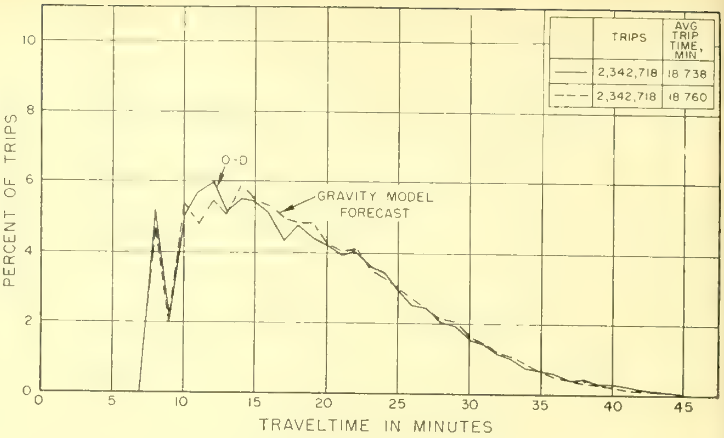

Figure 8.—Trip length distribution for total trips for all purposes from O-D survey and gravity model, Washington, D.C., 1955.

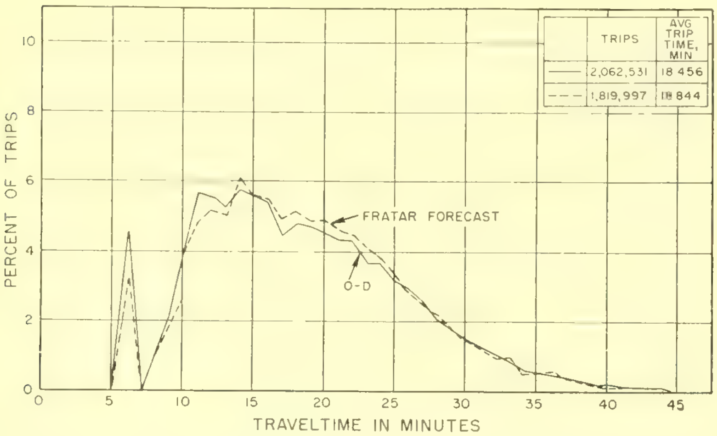

Figure 7.—Trip length distribution for total trips from O-D survey and Fratar model, Washington, D.C., 1955.

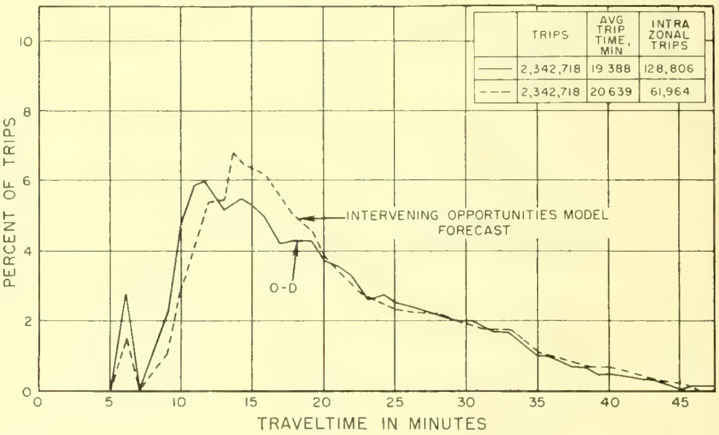

Figure 9.—Trip length distribution for total trips for all purposes from O-D survey and intervening opportunities model, Washington, D.C., 1955.

The curves in figure 6 for the competing opportunities model are for work trips only. A full analysis of this procedure could not be made because of calibration problems. The information in figure 6 represents the best calibration achieved with this procedure.

As expected, the gravity model had the best agreement through most portions of the trip length frequency curves because of the refined degree of adjustment made during the calibration phase. Both the gravity and intervening opportunities models produced good duplication of the total hours of travel and average trip length.

Even though the two curves for the competing opportunity model (fig. 6) have some agreement, no rational method could be found to adjust toward a more satisfactory model.

Forecast year

The trip length frequency curves from the travel patterns as estimated by each of the procedures are shown for comparison to the appropriate O–D information in figures 7–9. No forecast was made for the competing opportunities model.

The Fratar procedures provided a good duplication of average trip length for 1955 as shown in figure 7, although approximately 195,000 trips out of the total available 2,012,947 trips were not distributed because of zero trip ends for certain purposes in particular zones in 1948. The average trip length of the expanded patterns for 1955 of 18.8 minutes compares favorably with that of 18.5 minutes obtained from the surveyed information.

Travel patterns forecast with the gravity model likewise provided an extremely good duplication of the average trip length, as well as close agreement with the trip length frequency curve (fig. 8). The average traveltime for the forecast gravity model results of 18.8 minutes compared very well with the 18.7 minutes from the surveyed data. The intervening opportunities model forecast is shown in figure 9. The average traveltime (driving time plus terminal times) of 20.6 compared well with the actual traveltime of 19.4. These figures included the use of a river impedance.

River crossings

The tests of estimated river crossings made on the different model results were developed because of definite bias in the simulated trip distributions of two of the models, which became apparent during the calibration stage of model development. The use of time penalties on the Potomac River in the base year and in the forecast year was required for both the gravity model and the intervening opportunities model. Different impedances were required for the two models. The gravity model research was completed first and procedures to forecast these time penalties were developed at that time. When these procedures were applied during the intervening opportunities research, the error in the forecast year was reduced substantially but not completely. The penalties required in the gravity model and intervening opportunities model were different. The different methods required to forecast the time penalties is probably related to the different manner in which time is used by each model. Of course, the effect of the impedance to free travel in the form of the Potomac River bridges was present in the 1948 surveyed trip crossings, which were expanded to 1955 by the Fratar procedures. Table 2 data show the relative accuracies of river crossing estimates for the Potomac and Anacostia Rivers for each of the models for both the calibration and forecasting phases. The effect of the use of time impedances for the gravity and intervening opportunities model is included.

Movements by Corridor

A test on the movements by corridor to and from the CBD was developed to isolate any geographical bias present in model results. The incorporation and need for adjustment for

Table 2.—Total trips crossing the Potomac and Anacostia Rivers for comparison of data from Washington, D.C., O–D surveys, and models

Data source	Potomac River		Anacostia River	
	Trips	Difference from O–D data [1]	Trips	Difference from O–D data [1]
1948:	*Number*	*Percent*	*Number*	*Percent*
O–D survey	196, 255	NA	183, 696	NA
Gravity model	202, 237	+3. 05	184, 188	+0. 27
Intervening opportunities model	188, 134	−4. 14	193, 398	+5. 28
1955:				
O–D survey	246, 268	NA	287, 452	NA
Fratar model [2]	279, 055	+13. 31	281, 881	−1. 94
Gravity model	230, 949	−6. 22	296, 830	+3. 26
Intervening opportunities model	287, 447	+16. 72	318, 269	+10. 72

[1] Survey data used as base.
[2] Adjusted to common O–D survey base.

Table 3.—Calibration accuracy of different mathematical models, Washington, D.C., 1948

Sector	Travel between zero sector and sector number				
	O–D survey trips	Gravity model [1]		Intervening opportunities model [2]	
		Trips	Difference from O–D data	Trips	Difference from O–D data
Number	*Number*	*Number*	*Percent*	*Number*	*Percent*
0	134, 951	141, 105	+4. 56	142, 595	+5. 66
1	44, 771	46, 110	+2. 99	45, 407	+1. 42
2	72, 206	66, 494	−7. 91	59, 710	−17. 31
3	195, 114	181, 860	−6. 79	184, 815	−5. 28
4	93, 542	92, 027	−1. 62	94, 923	+1. 48
5	62, 484	58, 550	−6. 30	64, 999	+4. 02
6	80, 275	83, 684	+4. 25	91, 174	+13, 58
7	67, 835	68, 898	+1. 57	58, 299	−14. 06
8	42, 833	43, 505	+1. 57	36, 297	−15. 26
TOTAL	794, 011	782, 233	−1. 48	778, 219	−1. 99

[1] Includes *K* factors.
[2] Does not include *K* factors.

Table 4.—Forecasting accuracy of different mathematical models in duplicating home interview data, Washington, D.C., 1955

Sector	Travel between zero sector and sector number							
	O–D survey trips	Gravity model [1]		Intervening opportunities model [2]		O–D survey trips [3]	Fratar model [3]	
		Trips	Difference from O–D data	Trips	Difference from O–D data		Trips	Difference from O–D data
Number	*Number*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Number*	*Percent*
0	112, 471	123, 243	+9. 58	119, 613	+6. 35	112, 007	113, 972	+1. 75
1	52, 391	53, 830	+2. 75	53, 680	+2. 46	52, 213	47, 485	−9. 06
2	100, 710	87, 896	−12. 72	82, 498	−18. 08	88, 865	79, 388	−10. 66
3	197, 167	182, 558	−7. 41	187, 026	−5. 14	191, 362	181, 933	−4. 93
4	102, 384	105, 913	+3. 48	108, 668	+6. 14	97, 906	98, 860	+0. 97
5	64, 788	62, 019	−4. 27	70, 485	+8. 79	64, 623	63, 348	−1. 97
6	95, 461	100, 579	+5. 36	107, 037	+12. 13	92, 087	84, 960	−7. 74
7	69, 221	64, 911	−6. 23	66, 541	−3. 87	62, 125	62, 161	+0. 06
8	57, 847	54, 652	−5. 52	53, 258	−7. 93	51, 154	49, 653	−2. 93
TOTAL	852, 440	835, 631	−1. 97	848, 806	−0. 43	812, 342	781, 760	−3. 76

[1] Includes *K* factors.
[2] Does not include *K* factors.
[3] Contains information from 362 zones only, as used in Fratar model.

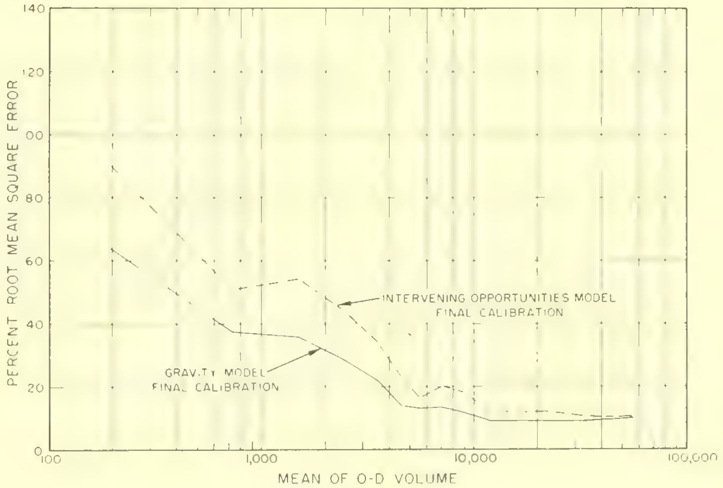

Figure 10.—RMSE for total trips for all purposes by volume groups from final calibration of gravity model and intervening opportunities model, Washington, D.C., 1948.

geographical bias has been shown for the gravity model through the use of *K* factors. No such adjustments were used in the Fratar or intervening opportunities procedures. Tables 3 and 4 contain information to relate the estimated patterns to and from the CBD, by corridor, to the actual patterns obtained from the O–D surveys for 1948 and 1955. In the gravity model, factors to adjust for geographical bias were for the work trips to the CBD.

Statistical Analysis of Assigned Trips

As a common measure of the accuracy of each of the model distributions, the total person trip output for the calibration and forecast runs of each model were assigned to a spider network and compared by link with the O–D survey assigned to the same network. All trips were defined as going from origin zone to destination zone. To achieve uniformity, the gravity model trips were redefined. Standard gravity model procedures were used to adjust the production to attraction trip tables to true origin to destination trip tables for directional assignments. To do this a 50–50 split of all production to attraction zone-to-zone transfers was assumed to get back to true origin to destination tables. For example, in determining the number of trip productions and trip attractions in any zone, the home end of any home based trip is always called the production and the nonhome end is always called the attraction. All trips with the general purpose of work would be considered as going from home to work, the work to home portions being reversed. After the model simulates trips by this definition, again assuming work trips, all home based trips are then converted to directional volumes by assuming 50 percent are trips from work to home.

Comparisons were made of directional link volumes assigned to a spider network and the differences recorded by volume group. Statistical analyses were made of these comparisons with the RMSE calculated for each model for each O–D volume group. The results of these analyses for the calibration of base year gravity and intervening opportunities models are shown in figure 10. Each model output includes the river time penalties. In the gravity model, *K* factors were used to adjust the work trips to the CBD. Next, RMSE by volume group for the forecast travel patterns for each of the models is shown in figure 11. The Fratar output is shown in relation to the O–D from only the 362 zones where compatibility for 1948 and 1955 could be achieved.

The analyses showed that the gravity model forecasts were closest to the O–D data in most volume groups up to 1,500 trips; the Fratar procedure and the intervening opportunities model had slightly better agreement in the very highest volume groups. River impedances were used with both the gravity and intervening opportunities models. However, the opportunities model could not be adjusted as closely as the gravity model to actual river crossings.

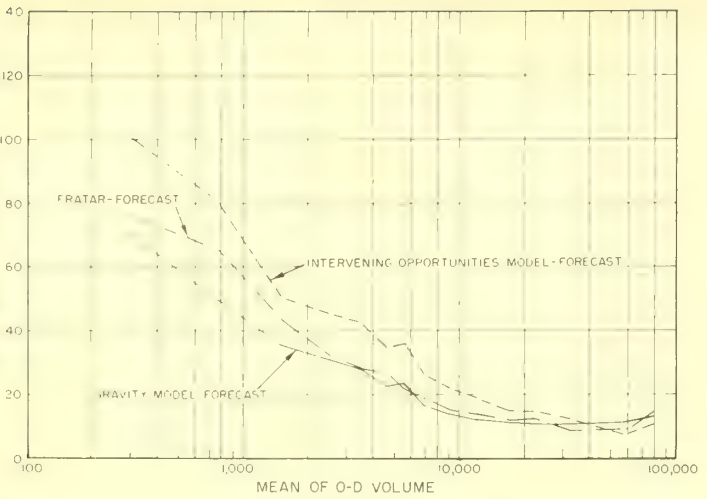

Figure 11.—RMSE for total trips for all purposes by volume groups forecast by Fratar, gravity, and intervening opportunities models, Washington, D.C., 1955.

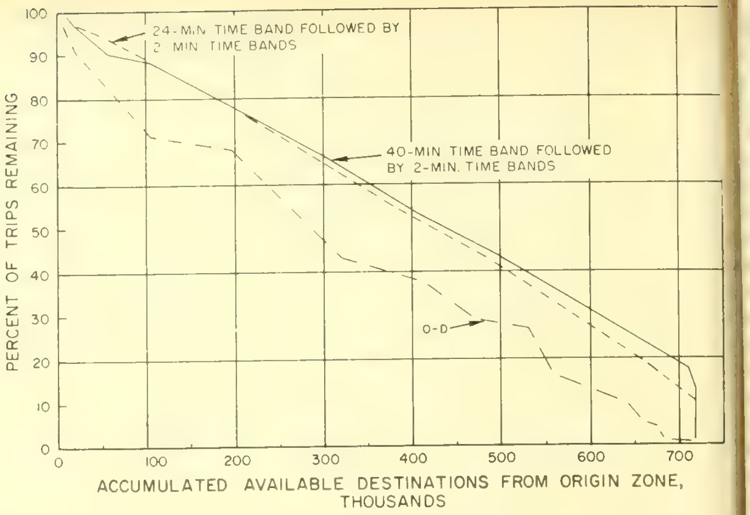

Figure 12.—Distribution of 10,687 home based work trips from zone 48 from O-D survey and by time bands from competing opportunities model, Washington, D.C., 1948.

The results of trip distributions by the Fratar procedure were biased in that 195,000 trips, for one cause or another, could not be expanded. It might be expected that the Fratar procedures would produce results that would have increasing error as the forecast period was lengthened and land use changes increased in significance but, even over such a relatively short time as 7 years, the Fratar forecasts were not significantly better than the other model results.

Analysis

An attempt was made to test on a common basis the four procedures being used to distribute and forecast urban travel patterns. When large masses of data and a series of formulations requiring different definitions and calibration procedures are used, variations in the base conditions occur. These variations in the test conditions did not seriously detract from the analysis of the relative merits and weaknesses of each of the procedures.

Fratar procedure

The Fratar procedure requires no calibration and performed essentially as expected. Six trip purposes were used: home, work, shop, social-recreational, school, and miscellaneous. Over the 7-year period, the results from the Fratar procedure had a high level of accuracy in all analytical tests. However, the Fratar procedure was not tested specifically in a most critical area—the correct expansion of trips from zones that are changing from essentially undeveloped rural land uses to full urban development. Most zones in this class had to be eliminated from the analysis because of the incompatibility of 1948 and 1955 zone boundaries. The model by its nature does not require any type of adjustment because of the socio-economic trip linkages inherent in the travel patterns expanded. It was surprising that the Fratar procedure was only moderately better, in estimating trips to and from each of the eight sectors and the CBD, than the gravity and intervening opportunities models. This particular test is the most sensitive indicator of socio-economic bias.

The multipurpose Fratar procedure has some distinct advantages in the proper expansion of trips by purpose but also has certain drawbacks when compared with a single purpose Fratar. By expanding the number of trip categories to six, the possibility of zero volumes in the trip tables increases. In the Washington area, 195,000 trips were lost in the expansion because no trips had been made in 1948 in certain zones and certain trip categories. But in 1955, in the same zones and for the same trip purposes, 195,000 trips were made. This amounted to almost 10 percent of the 1955 trips. Had it been possible to include all fringe area zones in the analysis, the magnitude of the problem would have been much more serious. Again the most serious problems in forecasting occur for the urban fringe areas where, for example, shopping centers and golf courses are developed on farm or vacant land. It is not possible to achieve correct trip distributions for such areas unless base year trips are first synthesized for these areas with an interarea travel formula and then artificially superimposed on the base year travel pattern before the Fratar expansion is made.

Gravity model

The gravity model proved adequate in most respects. The calibration phases are particularly strong. Its orderly procedure allowed fine adjustments in the traveltime factors and the direct adjustment for socio-economic or geographic bias. The traveltime factors were stable over the 7-year period. One problem inherent in the gravity control procedure was the necessity for socio-economic adjustment factors—34 adjustment factors ranging from 2.23 to 0.29 were utilized. Developing relations between these factors and characteristics of the districts of residence or attraction can present problems in forecasting these characteristics. In Washington, D.C., the factors used to adjust work trips to the CBD were related to the average incomes of the residence zones. Another problem in use of the gravity model is the necessity for forecasting river impedances. These topographical impedances, which are probably related to historical deficiencies of capacity that include the complete lack of facilities, can be projected on the basis of present and projected volume-capacity ratios. River barriers are a problem because they require a detailed, though not complex analysis, and because they relate to such a critical area in terms of the analysis of future transportation system needs.

Intervening opportunities model

The intervening opportunities model, although not previously utilized operationally by the researchers, provided very good results. Several methods of calibration were tried and, after selection of the best procedures, the model was calibrated with little difficulty. No socio-economic adjustment factors were used. The trip purposes were defined by the intervening opportunities model so that directional trips were maintained at all times. Fairly large river impedances were required and, as with the gravity model, their projection was straight forward although detailed analysis was required. However, even with the projected river impedance of 8 minutes, this model overestimated the 1955 Potomac River crossings. Examination of the results and the skim trees indicated that very little additional improvement could have been made even with a higher impedance value. A drawback in the use of this model is the fact that the L values change with time. In the analysis reported here the change in L value was forecast as a function of the change in the number of trips. Refinement in methods of forecasting these L values will require refinements in methods to project future trip lengths. Such a projection was not attempted in the application reported here. Considerable research is currently underway on trip length trends. The University of Pennsylvania, Institute for Urban Studies in cooperation with the Urban Planning Division of the Bureau of Public Roads has recently completed a research project, *Trip Length Variations among Urban Activities* and is currently undertaking a second project, *Trip Length Variations among Urban Areas.*

An additional point for consideration is the fact that in the calibration phase, the intervening opportunities models for the individual trip purposes do not necessarily reproduce the trip length frequency characteristics of the corresponding O–D trips. When the individual purposes are summed to a total purpose the trip length frequency characteristics are good because of compensating deviations in the individual trip purposes. The explanation given for this situation is that in the opportunities model, trip purposes per se were not used but the survey trip purposes were used as a convenient way of grouping trip ends to apply individual *L* values. Problems may arise when trips are distributed by purpose; for example, when performing a modal split analysis. The *L* value derived to treat a single purpose would differ from the *L* value used if the trip purpose were to be combined with others to form a total trip distribution. In essence, the trip distributions by purpose were meaningful only when summed to a total trip purpose distribution.

Competing opportunities model

The fact that the competing opportunities model could not be calibrated with the Washington, D.C., O–D data and on a zonal basis was disappointing. Time bands of uniform width were not applicable and no simple procedure could be derived for selecting nonuniform time bands. Many different combinations of time bands were tried before a set was obtained that even approached providing correct trip length characteristics. When the different trial and error approaches of arriving at appropriate time bands proved futile, a theoretical approach to the problem was attempted. The required type of probability curve for selected Washington, D.C. zones was derived. It was a plot of the percent of trips remaining to be distributed related to the accumulated available opportunities. Working within the framework of the model, it was not possible to duplicate the probability curve derived from the selected zonal O–D data. Figure 12 illustrates the degree to which two different time band groupings approach the actual O–D probability groupings.

Future research

Several areas for future research were determined when the models were analyzed on a common basis. The use of different trip purpose categories as input to the gravity model trip distribution procedure should be explored as a means of eliminating the socio-economic adjustment factors. As a first attempt, a five-purpose, true origin and destination purpose definition model consisting of (1) home to work trips, (2) work to home trips, (3) home to other trips, (4) other to home trips, and (5) nonhome based trips should be tried.

Research is needed to develop more sophisticated procedures to adjust the base year *L* values to the future year for the intervening opportunities model. Certainly, better information on future trip length in terms of either miles or minutes would be very helpful. Also, some work is required to test the effect of the trip universe used on accuracy and the need to make adjustments to force all the trips to be sent. For example, the inclusion or exclusion of the external trip ends creates a slightly different set of intervening opportunities for any given origin zone.

Additional research is also needed in which the impedance effect on travel of physical or topographical features would be studied. More insight into basic causes of the impedance is essential for the development of comprehensive techniques for projecting the impedance. The advantages of the purpose Fratar model, that is the more direct consideration of land use changes, must be investigated in relation to the resultant and very significant loss in expanded trips. Finally research is required to develop calibration procedures for the competing opportunities model.

Summary

The overall accuracy of the gravity model was slightly better than the accuracy of the intervening opportunities model in base year simulation and in forecasting. But more parameters were used in the gravity model calibration. When socio-economic adjustment factors were used, the gravity model test results had less error than the intervening opportunities model when trips by sector to the CBD were examined. However, better results were obtained from the opportunities model than from the unadjusted gravity model. The cause of this difference in results from the two models is not definite but may have been either the conceptual basis of the models or the trip purpose stratifications used. The intervening opportunities model was slightly less difficult to calibrate than the gravity model because fewer parameters were used. However, adjustments required for future *L* values reduced this advantage in making the forecasts. The gravity and intervening opportunities models proved to be about equal in reliability and utility in simulating the 1948 and 1955 trip distributions for Washington, D.C.

The Fratar growth factor procedure was useful in correctly expanding trips for stable areas but had significant weaknesses related to areas undergoing land use changes. The concentration of error for areas experiencing growth in trips emphasizes the need for supplemental procedures to provide a base year synthesized trip pattern for such areas. The magnitude of this problem in relation to the favorable results attained with the gravity and intervening opportunities models indicates that the use of a travel model provides a more direct and efficient approach to trip distribution for growing urban areas. It was not possible to adequately calibrate the competing opportunities model for use in determining trip distributions between areas as small as the traffic zones used in Washington, D.C.

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Two new publications have recently been completed by the Bureau of Public Roads; both may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, prepaid. A brief description of each publication and its purchase price is given in the following paragraphs.

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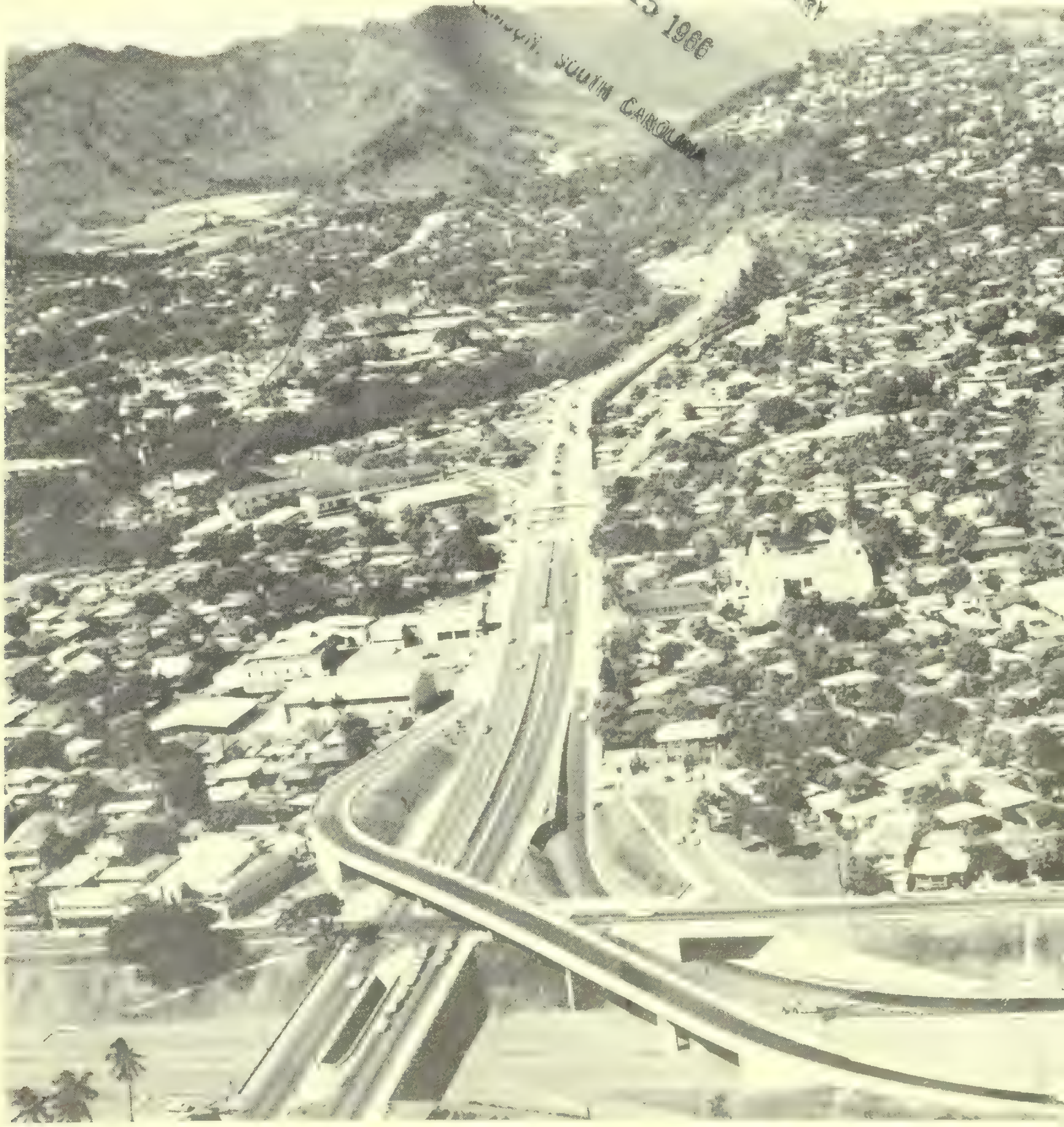

Pali Highway, Honolulu, Hawaii

Pali Highway is Federal Aid Primary Route 61 and runs across the island of Oahu from the harbor in Honolulu to connect with FAS 630 at Kailua. The connection with Lunalilo Freeway, Hawaii Interstate Highway 1, and the construction on this highway is shown in the foreground.

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Contents of this publication may be reprinted. Mention of source is requested.

Perceptual Basis of Vehicular Guidance

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

by **DONALD A. GORDON**,[1] **Research Psychologist,**
Traffic Systems Research Division

INTRODUCTION

Vehicular guidance is related to the driver's visual environment—described in basic terms of position and movement—in the four parts of this article: "Generalized Equations, the Driver's Moving Visual Environment"; "Static and Dynamic Visual Fields in Vehicular Guidance"; "Motion Parallax and Perceptual Hypothesis Testing"; and "Perceptual Mechanisms in Vehicular Guidance." As the visual environment is an organized spatial entity, hence it may be considered a field having static and dynamic aspects. The positional field includes the angular coordinates of spatial points around the eye position. The velocity field includes the vectors of angular motion around the driver's eyes, as he moves on his path. These vectors vary with the speed and direction of the driver's motion. Inasmuch as the location of the vectors is determined by the positional field, this field might be called the positional-velocity field. The acceleration field is defined in terms of angular acceleration vectors rather than velocity. This field might be called the positional-acceleration field.

In Part 1, equations are presented on the general organization of visual space around the eye of the moving driver. Derivations included on the effects of rectilinear motion, and horizontally and vertically curved motions, and combinations of these motions. In Part 2, the principles applying to the perception of the positional, velocity, and acceleration fields under rectilinear motion are discussed. In Part 3, the concept of motion parallax, widely accepted as a cue to depth, is examined. It was concluded that terrain movements on a circular path could not be interpreted in any consistent manner to show the distance of seen objects. A perceptual hypothesis principle is proposed to explain the major contributions of observer motion to space perception. In Part 4, the driver's perceptions are analyzed in the basic vehicular maneuvers of steering, perceptual anticipation, and car following.

Findings and Conclusions

The perceptual problems in vehicular guidance were considered in the context of the positional, velocity, and acceleration fields around the moving vehicle. These are very general and persistent aspects of the driver's visual environment. The equations governing these fields, and the fields themselves, were considered for features and regularities that might explain human spatial perception. The following conclusions were made from the analyses.

Part 2

- The interpretive scaling of visual angle is a key factor in interpreting perspective and in perceiving size, distance, and motion perception.
- Simple and obvious features of the visual environment, which have often been ignored in explanations of space perception, are believed to provide important aids for vehicular guidance. The roadway ahead of the vehicle, for example, may be used to obtain the scale of the terrain and objects in it.
- The driver may see his vehicle, or some part of the environment as reference for motion. If the foreground is visually fixated, a curious illusion of motion is seen. The background seems to rotate forward and around the foreground. This velocity parallax curl is based upon the difference in velocity vectors between foreground and background.
- Roadway boundaries and lane markings are used in alining the moving vehicle with the road. This conclusion challenges the often quoted statement that the focus of expansion is the cue for the direction of sensed locomotion.
- Angular acceleration increases as the square of vehicular speed. The consequences of this relation for the perception of vehicular speed are indicated.
- The pattern of the angular acceleration field does not resemble any familiar pattern of visual experience. It therefore seems that angular acceleration is not directly sensed. By extension, it is doubtful that higher derivatives of motion are seen as such.

Part 3

Helmholtz's formulation of the motion parallax cue to distance fails when the observer follows a curved path. Angular velocity on the ground plan does not decrease systematically with distance; rather it shows an asymmetrical pattern, which would lead to an erroneous interpretation under the rules of linear motion parallax. It has been suggested that observer motion aids space perception by providing a test of prior perceptual interpretations or hypotheses under changed perspective.

Part 4

- When the vehicle is alined with a straight or regularly curved highway, the road assumes a steady state appearance. The borders and lane markers remain almost stationary in the driver's field of view. The driver's problem in lateral guidance, car following, and other maneuvers may be to maintain an acceptable steady state condition and to null deviations from the steady condition by utilizing visual feedback information.
- If the moving vehicle is misalined laterally with the road, the entire field moves as a unit. No one part of the road borders or lane markers is essential for steering.
- The extent of lateral misalinement is indicated by the rate and extent of slewing and sideslipping of the road borders and lane markers. The driver's perceptual response is based upon an integration of these and other items of information.
- The driver's anticipation requirements must be considered in the design of road features such as curves, signal lights and signs. These requirements have not been extensively studied.
- The driver's ability to estimate the time required to reach an object ahead may be based upon his estimate of perceived distance.
- The visual stimulus to car following on a curvilinear path is discussed. It is concluded that need exists for empirical validation studies of car following theory.
- Guidance theories based upon characteristics of the velocity field, such as the motion parallax cue to depth, the center of expansion, and null locus indicators of alinement, fail on curved roads. Features of the velocity field shift with the vehicle's curved path of motion.

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[1] The author acknowledges the advice and support provided by Dr. Richard M. Michaels, now Science Advisor, Office of Research and Development, during the research.

Part 1—Generalized Equations—the Driver's Moving Visual Environment

Introduction

TO ANALYZE DRIVING, the moving visual environment to which the driver is reacting must be described. Equations describing the environment of linear translation at constant speed have been developed by other researchers (*1*, *2*).[2] This article presents a general procedure applicable to horizontal and vertical curvature, separately or in combination, with fixed or varying speeds of vehicles motion. To describe the moving environment, the position and induced movement of objects expressed in Cartesian coordinates (x, y, z, dx/dt, dy/dt, dz/dt) must be translated into the spherical coordinates (θ, ϕ, $d\theta/dt d\phi/dt$, $d^2\theta/dt^2$, $d^2\phi/dt^2$) of the observer's visual world.

Visual Coordinates

The coordinate system is shown in figure 1. The eye is at the origin. Distance ahead of the eye is represented by x, to the side by y, and up and down by z. Distance from the eye to a terrain or other point is represented by ρ whose projection on the xy plane is r. It is not essential that the ground surface be parallel to the momentary path. Thus, the ground surface relative to a landing aircraft can be specified in x, y, and z coordinates, and can be dealt with.

Angular position

Angular position can be determined by use of the following equations.

$$\theta = \arctan y/x \text{ rad.} \qquad (1)$$

$$\phi = \arcsin z/\rho \text{ rad.} \qquad (2)$$

Therefore,

$$r = (x^2 + y^2)^{1/2} \qquad (3)$$

$$\rho = (x^2 + y^2 + z^2)^{1/2} \qquad (4)$$

Angular velocity

To determine angular velocity the following procedure can be used.

$$\frac{d\theta}{dt} = \frac{1}{r^2}\left(-y\frac{dx}{dt} + x\frac{dy}{dt}\right) \text{ rad./sec.} \qquad (5)$$

$$\frac{d\phi}{dt} = \frac{1}{r}\left(\frac{-zx}{\rho^2}\frac{dx}{dt} - \frac{zy}{\rho^2}\frac{dy}{dt} + \frac{r^2}{\rho^2}\frac{dz}{dt}\right) \text{ rad./sec.} \qquad (6)$$

Angular acceleration

The equations for angular acceleration are, as follows.

$$\frac{d^2\theta}{dt^2} = \frac{2xy}{r^4}\left[\left(\frac{dx}{dt}\right)^2 - \left(\frac{dy}{dt}\right)^2\right] - \frac{1}{r^2}\left(y\frac{d^2x}{dt^2} - x\frac{d^2y}{dt^2}\right) + 2\left[\frac{y^2 - x^2}{r^4}\left(\frac{dx}{dt}\frac{dy}{dt}\right)\right] \text{ rad./sec./sec.} \qquad (7)$$

$$\frac{d^2\phi}{dt^2} = \frac{z}{\rho^4 r^3}\left[(\rho^2y^2 - 2r^2x^2)\left(\frac{dx}{dt}\right)^2 + (\rho^2x^2 - 2r^2y^2)\left(\frac{dy}{dt}\right)^2\right] - \frac{2rz}{\rho^4}\left(\frac{dz}{dt}\right)^2 + \left(\frac{z^2 - r^2}{\rho^4 r}\right)\left(2x\frac{dx}{dt}\frac{dz}{dt} + 2y\frac{dy}{dt}\frac{dz}{dt}\right) + \frac{2xyz}{\rho^4 r^3}(2r^2 + \rho^2)\frac{dx}{dt}\frac{dy}{dt} - \frac{zx}{r\rho^2}\frac{d^2x}{dt^2} - \frac{zy}{r\rho^2}\frac{d^2y}{dt^2} + \frac{r}{\rho^2}\frac{d^2z}{dt^2} \text{ rad./sec./sec.} \qquad (8)$$

Rectilinear Motion

When the eye moves with constant velocity in a straight line, the environment translates in x at a rate of $-dx/dt$ (negative speed of forward motion of the eye). The terms dy/dt and dz/dt equal zero. The equations of velocity and acceleration in spherical coordinates reduce to:

$$\frac{d\theta}{dt} = \frac{-y}{r^2}\frac{dx}{dt} \text{ rad./sec.} \qquad (9)$$

$$\frac{d\phi}{dt} = \frac{-zx}{\rho^2 r}\frac{dx}{dt} \text{ rad./sec.} \qquad (10)$$

$$\frac{d^2\theta}{dt^2} = \frac{2xy}{r^4}\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (11)$$

$$\frac{d^2\phi}{dt^2} = -z\left[\frac{\rho^2y^2 - 2r^2x^2}{\rho^4 r^3}\right]\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (12)$$

Horizontal Curvature

On a horizontally curved path, the environment shows dx/dt and dy/dt components, while dz/dt equals zero. It is convenient to describe the rotary movement relative to an origin distance, R distance in y from the observer's eye, as shown in figure 2. For point P, in figure 2, the following expressions apply.

β = positive angle of rotation of O_1P
$x_i = x$
$y_i = R - y$
$r_i = [(x_i)^2 + (y_i)^2]^{1/2}$

Then,

$$\overrightarrow{PQ} = \text{velocity vector of } O_1P = r_i\frac{d\beta}{dt} \qquad (13)$$

$$\frac{dx_i}{dt} = x_i \text{ component of } \overrightarrow{PQ} = r_i \sin\beta\frac{d\beta}{dt} = -y_i\frac{d\beta}{dt} = (y - R)\frac{d\beta}{dt} \text{ ft./sec.} \qquad (14)$$

$$\frac{dy_i}{dt} = y_i \text{ component of } \overrightarrow{PQ} = r_i \cos\beta\frac{d\beta}{dt} = x_i\frac{d\beta}{dt} = x\frac{d\beta}{dt} \text{ ft./sec.} \qquad (15)$$

$$\frac{d^2x_i}{dt^2} = -x\left(\frac{d\beta}{dt}\right)^2 + (y - R)\frac{d^2\beta}{dt^2} \text{ ft./sec./sec.} \qquad (16)$$

$$\frac{d^2y_i}{dt^2} = (y - R)\left(\frac{d\beta}{dt}\right)^2 + x\frac{d^2\beta}{dt^2} \text{ ft./sec./sec.} \qquad (17)$$

Substitution of dx_i/dt and $-dy_i/dt$ for dx/dt and dy/dt in equation (5) produces equation (18), which gives angular velocity at the driver's eye related to position and angular speed around origin, O_1.

$$\frac{d\theta}{dt} = \left[\frac{-y}{r^2}(y - R) - \frac{x}{r^2}(x)\right]\frac{d\beta}{dt} \text{ rad./sec.} \qquad (18)$$

This reduces to

$$\frac{d\theta}{dt} = \left(\frac{-y}{r^2} + \frac{1}{R}\right)\frac{dx}{dt} \text{ rad./sec.} \qquad (19)$$

On a curve to the right

$$\frac{d\theta}{dt} = \left(\frac{-y}{r^2} - \frac{1}{R}\right)\frac{dx}{dt} \text{ rad./sec.} \qquad (20)$$

Similar substitution in equation (6) produces

$$\frac{d\phi}{dt} = \frac{-z}{r\rho^2}[x(y - R) - xy]\frac{d\beta}{dt} = \frac{-xz}{r\rho^2}\frac{dx}{dt} \text{ rad./sec.} \qquad (21)$$

Angular Acceleration

Angular acceleration at the driver's eye related to position and angular acceleration around origin, O_1, can be derived from equations (7) and (8) by substitution of dx_i/dt, $-dy_i/dt$, d^2x_i/dt^2, $-d^2y_i/dt^2$ for dx/dt, dy/dt, d^2x/dt^2, d^2y/dt^2 and by setting $d^2\beta/dt^2$ equal to zero.

$$\frac{d^2\theta}{dt^2} = \left(\frac{2xyR^2}{r^4} - \frac{xR}{r^2}\right)\left(\frac{d\beta}{dt}\right)^2 = \left(\frac{2xy}{r^4} - \frac{y}{Rr^2}\right)\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (22)$$

On a curve to the right

$$\frac{d^2\theta}{dt^2} = \left(\frac{2xy}{r^4} + \frac{x}{Rr^2}\right)\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (23)$$

$$\frac{d^2\phi}{dt^2} = \left[\frac{-z}{\rho^4 r^3}(\rho^2y^2 - 2r^2x^2) + \frac{zy}{\rho^2 rR}\right]\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (24)$$

On a curve to the right

$$\frac{d^2\phi}{dt^2} = \left[-\frac{z}{\rho^4 r^3}(\rho^2y^2 - 2r^2x^2) - \frac{zy}{\rho^2 rR}\right]\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \qquad (25)$$

Vertical Curvature

The analysis of vertical curvature (see fig. 3) follows that of horizontal curvature. Environmental movement is resolved into dx/dt and dz/dt components; dy/dt movement is absent, and hence equal to zero. In figure 3, the eye is at the origin, A feet above the ground. The center of rotation is at a distance R above the ground (below for a convex curvature). There are two derivations: concave (upward) and convex (downward).

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[2] The numbers in parentheses indicate the references listed on page 67.

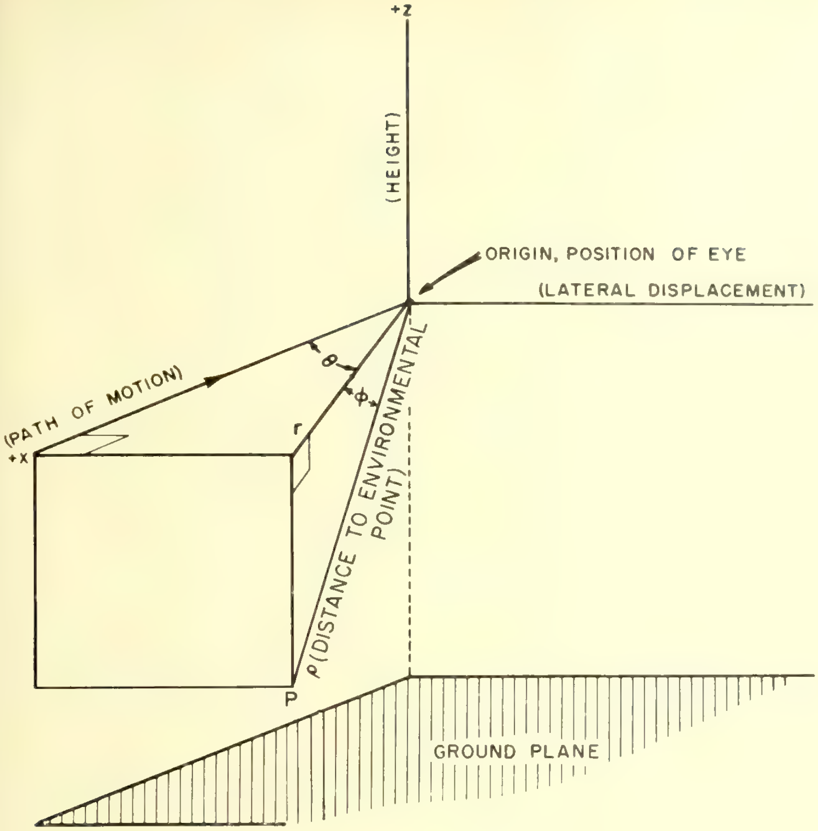

Figure 1.—Basic coordinate relationships.

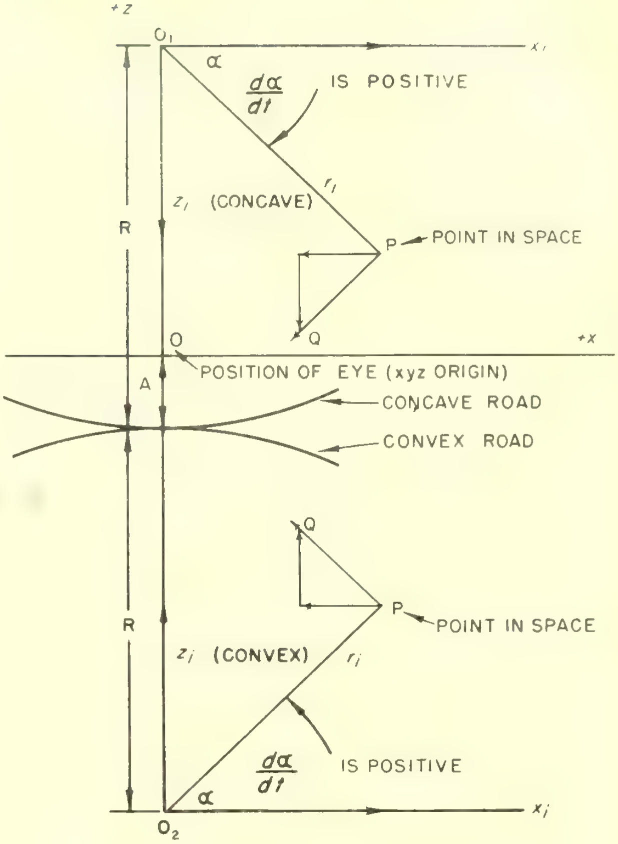

Figure 3.—Concave and convex vertical curvature.

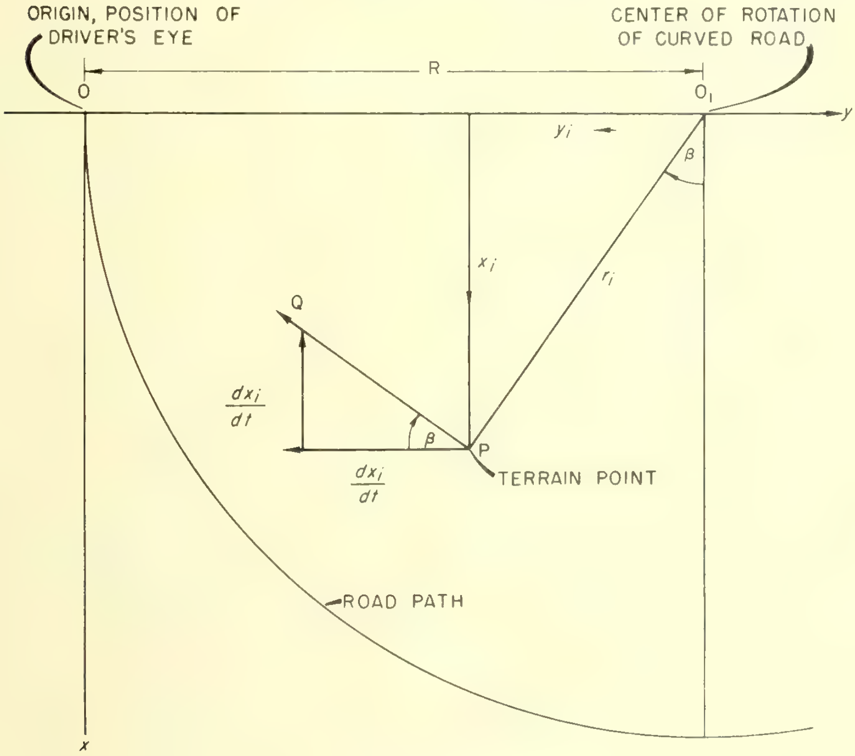
ORIGIN, POSITION OF DRIVER'S EYE
CENTER OF ROTATION OF CURVED ROAD
R
O
O1
y
yi
β
xi
ri
Q
dxi/dt
β
P
dxi/dt
TERRAIN POINT
ROAD PATH
x

Figure 2.—Horizontal curvature.

Concave curvature

For environmental point P, when the curvature is concave,

α=positive angle rotation of O_1P

$x_i=x$

$z_i=R-z-A$

$$r_i=[(x_i)^2+(z_i)^2]^{1/2}$$

$$\overrightarrow{PQ}=\text{velocity vector of } \overrightarrow{O_1P}=r_i\frac{d\alpha}{dt}$$

$$\frac{dx_i}{dt}=x_i \text{ component of } \overrightarrow{PQ}=-r_i\sin\alpha\frac{d\alpha}{dt}$$

$$=-z_i\frac{d\alpha}{dt}=-(R-z-A)\frac{d\alpha}{dt}\text{ ft./sec.}$$

$$\frac{dz_i}{dt}=z_i \text{ component of } \overrightarrow{PQ}=r_i\cos\alpha\frac{d\alpha}{dt}$$

$$=x_i\frac{d\alpha}{dt}=x\frac{d\alpha}{dt}\text{ ft./sec.}$$

$$\frac{d^2x_i}{dt^2}=-x\left(\frac{d\alpha}{dt}\right)^2-(R-z-A)\frac{d^2\alpha}{dt^2}\text{ ft./sec./sec.}$$

R and A are positive.

$$\frac{d^2z_i}{dt^2}=\frac{d}{dt}\left(r_i\cos\alpha\frac{d\alpha}{dt}\right)$$

$$=-(R-z-A)\left(\frac{d\alpha}{dt}\right)^2+x\frac{d^2\alpha}{dt^2}\text{ ft./sec./sec.}\tag{26}$$

To determine angular velocity for concave curvature use equations (1) and (2). When dx_i/dt and $-dz_i/dt$ are substituted for dx/dt and dz/dt and when dy/dt is set equal to zero, equations (5) and (6) reduce to

$$\frac{d\theta}{dt}=\left\{-\frac{y}{r^2}\left[1-\frac{(z+A)}{R}\right]\right\}\frac{dx}{dt}\text{ rad./sec.}\tag{27}$$

$$\frac{d\phi}{dt}=\left[-\frac{zx}{r\rho^2}\left(1-\frac{(\rho^2+zA)}{zR}\right)\right]\frac{dx}{dt}\text{ rad./sec.}\tag{28}$$

Note that

$$\frac{d\alpha}{dt}=-\frac{1}{R}\frac{dx}{dt}\tag{29}$$

Angular accelerations for concave curvature related to rectilinear coordinates of motion can be derived from equations (7) and (8) by substitution and by setting $d\alpha^2/dt^2$ equal to zero.

$$\frac{d^2\theta}{dt^2}=\left[\frac{2xy}{r^4}+\frac{2xy}{R^2r^4}\left(z^2+A^2-2Rz-2AR\right.\right.$$

$$\left.\left.+2Az+\frac{r^2}{2}\right)\right]\left(\frac{dx}{dt}\right)^2\text{ rad./sec./sec.}\tag{30}$$

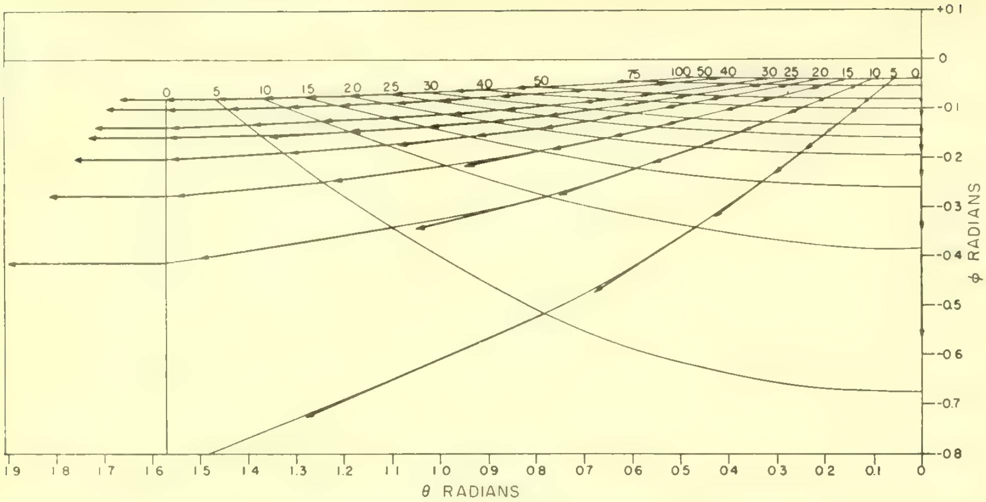

Figure 4.—Vector field of linear velocity shown on horizontal plane.

$$\frac{d^2\phi}{dt^2}=\left\{\frac{-z}{\rho^4r^3}(\rho^2y^2-2r^2x^2)+\frac{1}{R^2}\left[\frac{Ax^2z}{\rho^4r}\right.\right.$$
$$(2A-4R)+\frac{y^2z}{\rho^2r^3}(Rz+2AR-A^2-2Az)$$
$$\left.-Rx^2r^2\right]+\frac{A}{\rho^2r}(x^2-y^2)+\frac{y^2}{r^3}$$
$$\left.(R-z)\right\}\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \quad (31)$$

Convex curvature

For environmental point P, when the curvature is convex,

$\alpha=$ positive angle of rotation of O_2P
$x_i=x$
$z_i=R+z+A$
$r_i=[(x_i)^2+(z_i)^2]^{1/2}$
$\vec{PQ}=$velocity vector of $\overrightarrow{O_2P}=r_i\frac{d\alpha}{dt}$

$$\frac{dx_i}{dt}=x_i \text{ component of } \overrightarrow{PQ}=-r_i\sin\alpha\frac{d\alpha}{dt}$$
$$=-z_i\frac{d\alpha}{dt}=-(R+z+A)\frac{d\alpha}{dt}$$
$$\frac{dz_i}{dt}=z_i \text{ component of } \overrightarrow{PQ}=r_i\cos\alpha\frac{d\alpha}{dt}$$
$$=x_i\frac{d\alpha}{dt}=x_i\frac{d\alpha}{dt}$$

Therefore,

$$\frac{d^2x_i}{dt^2}=\frac{d}{dt}\left(-r_i\sin\alpha\frac{d\alpha}{dt}\right)=-r_i\cos\alpha\left(\frac{d\alpha}{dt}\right)^2$$
$$-r_i\sin\alpha\left(\frac{d^2\alpha}{dt^2}\right)$$
$$= x\left(\frac{d\alpha}{dt}\right)^2-(R+z+A)\frac{d^2\alpha}{dt^2} \text{ ft./sec./sec.} \quad (32)$$

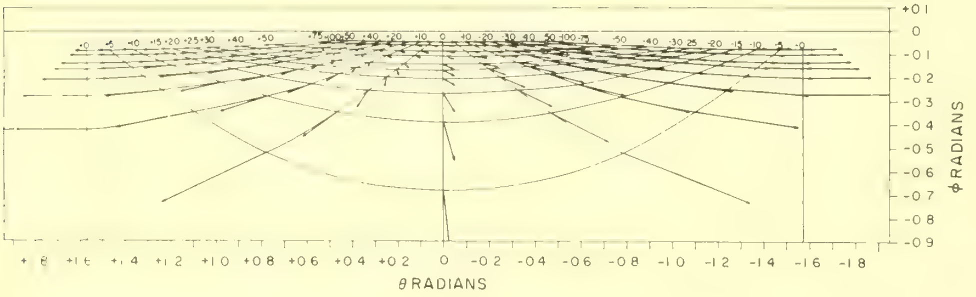

Figure 5.—Vector velocity field of horizontal curved motion shown on horizontal plane.

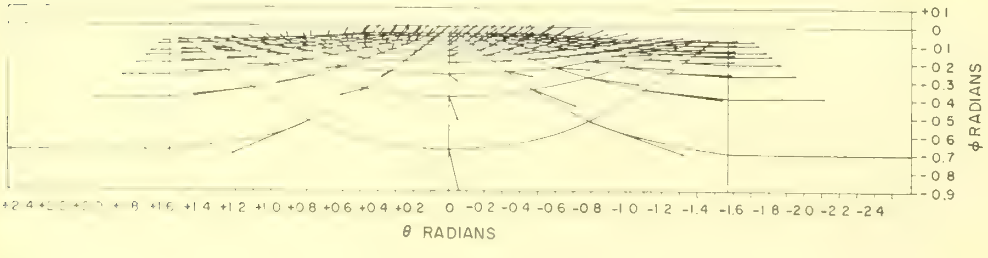

Figure 6.—Vector velocity field of horizontal and vertical curved motion shown on horizontal plane.

$$\frac{d^2z_i}{dt^2}=\frac{d}{dt}\left(r_i\cos\alpha\frac{d\alpha}{dt}\right)=-r_i\sin\alpha\left(\frac{d\alpha}{dt}\right)^2$$
$$+r_i\cos\alpha\left(\frac{d^2\alpha}{dt^2}\right)$$
$$=-(R+z+A)\left(\frac{d\alpha}{dt}\right)^2+x\frac{d^2\alpha}{dt^2} \text{ ft./sec./sec.} \quad (33)$$

Angular position, velocity, and acceleration

The derivations for angular velocity and acceleration for convex curvature parallel those for concave curvature.

$$\frac{d\theta}{dt}=\left\{\frac{-y}{r^2}\left[1+\left(\frac{z+A}{R}\right)\right]\right\}\frac{dx}{dt} \text{ rad./sec.} \quad (34)$$

$$\frac{d\phi}{dt}=\left\{\frac{-z}{r\rho^2}\left[1+\left(\frac{\rho^2+zA}{zR}\right)\right]\right\}\frac{dx}{dt} \text{ rad./sec.} \quad (35)$$

$$\frac{d^2\theta}{dt^2}=\left[\frac{2xy}{r^4}+\frac{2xy}{r^4R^2}\left(z^2+A^2+2Rz+2AR\right.\right.$$
$$\left.\left.+2Az+\frac{r^2}{z}\right)\right]\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \quad (36)$$

$$\frac{d^2\phi}{dt^2}=\left\{\frac{-z}{\rho^4r^3}(\rho^2y^2-2r^2x^2)+\frac{1}{R^2}\left[\frac{Ax^2z}{\rho^4r}(2A+4R)\right.\right.$$
$$+\frac{y^2z}{\rho^2r^3}(-Rz-2AR-A^2-2Az)$$
$$\left.\left.+Rx^2r^2+\frac{A}{\rho^2r}(x^2-y^2)\frac{-y^2}{r^3}(R+z)\right]\right\}$$
$$\left(\frac{dx}{dt}\right)^2 \text{ rad./sec./sec.} \quad (37)$$

Complex curves

On complex curves, the angular velocity is the sum of horizontal and vertical curvature contributions. Acceleration components also summate. Angular position is unaffected by motion of the vehicle. For example, the angular velocity formulas for a left convex curve can be developed from combinations of previous equations. R_1 is the horizontal radius of curvature and R_2 is the vertical radius.

$$\frac{d\theta}{dt}=\left[\frac{-y}{r^2}+\frac{1}{R_1}-\frac{y}{r^2}\frac{(z+A)}{R_2}\right]\frac{dx}{dt} \text{ rad./sec.} \quad (38)$$

$$\frac{d\phi}{dt}=\left[\frac{-yz}{r\rho^2}-\left(\frac{\rho^2+zA}{zR_2}\right)\right]\frac{dx}{dt} \text{ rad./sec.} \quad (39)$$

If the motion of the vehicle varies in velocity, an additional term is required in the angular acceleration equations—positional and velocity equations are unaffected by acceleration. In the linear situation, the additional terms for angular acceleration can be derived from equations (6) and (7), where the d^2x/dt^2 terms represent the presumably known acceleration.

$$\frac{d^2\theta}{dt^2}=\frac{2xy}{r^4}\left(\frac{dx}{dt}\right)^2-\frac{y}{r^2}\frac{d^2x}{dt^2} \text{ rad./sec./sec.} \quad (40)$$

$$\frac{d^2\phi}{dt^2}=\frac{-z}{\rho^4r^3}(\rho^2y^2-2r^2x^2)\left(\frac{dx}{dt}\right)^2$$
$$\frac{-zx}{r\rho^2}\frac{d^2x}{dt^2} \text{ rad./sec./sec.} \quad (41)$$

For defining angular acceleration for curved motion, the additional acceleration terms needed include d^2x_i/dt^2, d^2y_i/dt^2, and d^2z_i/dt^2. In horizontal curvature, the x_i component of acceleration has been defined in equation (16) as,

$$\frac{d^2x_i}{dt^2}=-x\left(\frac{d\beta}{dt}\right)^2 +(y-R)\,\frac{d^2\beta}{dt^2}\text{ ft./sec./sec.} \qquad (16)$$

Where the horizontal curved motion is uniform, the $d^2\beta/dt^2$ component of acceleration is zero. If the motion accelerates, this term is not zero, but is evaluated as

$$(y-R)\,\frac{d^2\beta}{dt^2}=\left(1-\frac{y}{R}\right)\frac{d^2x}{dt^2} \qquad (42)$$

involving the relationship of

$$d^2\beta/dt^2=\frac{-d^2x/dt^2}{R} \qquad (43)$$

where d^2x/dt^2 is the acceleration of the eye or vehicle. Similarly, the acceleration terms of other motions can be developed. Velocity and acceleration fields having implications for the theory of human space perception are presented in figures 4 through 7. Additional information on the applications of velocity and acceleration fields is given in references *1* and *2*.

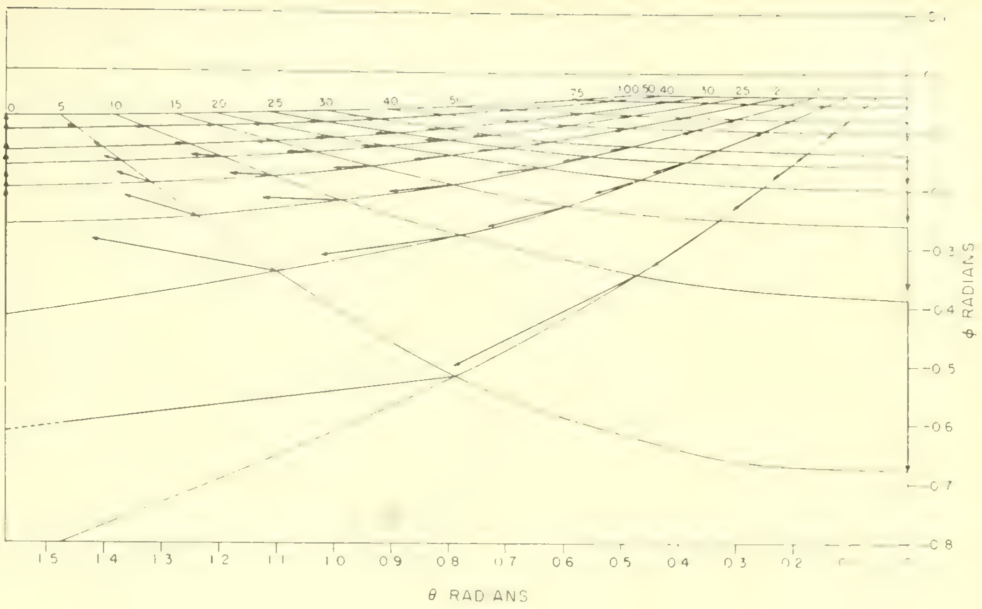

Figure 7.—Vector field of linear acceleration on horizontal plane.

In figure 4 the vector field of linear velocity is shown from a position 4 feet above the plane. The field is bilaterally symmetrical and the rear mirrors the front, with the vectors reversed. As shown, the velocity vectors are seen as being tangential, and sometimes coincident, with the flow lines of angular positions. The vector velocity field of horizontal curved motion on the horizontal plane is also shown from a position 4 feet above the plane. The radius of curvature is 100 feet to the left of the eye (origin). The field is asymmetrical and has a null point at the center of curvature, which approaches a value of $1/R\ dx/dt$ at infinite distance.

The vector velocity field for both horizontal and vertical curved motion is shown on the horizontal plane in figure 6. As in figures 4 and 5 the field is shown from a position 4 feet above the plane. The center of horizontal curvature is viewed 100 feet to the left of the origin, the center of vertical curvature is 100 feet below the eye. The field shown in figure 6 and in figure 5 have implications for perceptual theories that relate space perception to features of the velocity field, such as the motion parallax cue to distance. Under curved motion, features of the velocity field are too unstable to provide a basis for the perception of distance or orientation.

As in figures 4 through 6, the vector field of linear acceleration on the horizontal plane is shown in figure 7 from a position 4 feet above the plane. The basic psychological question raised by this field is whether acceleration information is immediately sensed as a change of velocity. The lack of resemblance of this field to any familiar pattern of visual experience provides evidence that angular acceleration is not directly sensed.

Part 2—Static and Dynamic Visual Fields in Vehicular Guidance

Research Approach

THE IMPORTANCE for understanding driving, of analyzing the visual input, can readily be recognized. The investigation of visual inputs reported in this article was begun with the study of the positional, velocity, and acceleration fields around the moving vehicle. These fields are general and persistent aspects of the visual environment. The velocity and acceleration fields, which present time varying aspects of the environment, are of particular interest as they provide information not available in static viewing.

The many cues available in spatial perception have been discussed by previous investigators (*3*, *4*, and *5*) and the terrain characteristics that may orient the human in his spatial environment have also been described (*6*). These studies indicate methods that might be employed in vehicular guidance; but research also should be aimed at identifying the inputs that the driver actually uses. Other researchers have concerned themselves with human errors in space perception such as the systematic overestimation of size in distant vision (*7*) and the hyperbolic distortion shown in the judgment of space in certain reduction situations (*8*). In the analyses reported here, the characteristic of the driver's judgment of space to be explained is its accuracy rather than its incompatability with physical space.

In the preceding Part 1, the mathematical description of the moving ground plane from the driver's point of view was developed. The environment seen by the driver involves a perspective transformation of ground position, velocity, and acceleration. The positional field includes the angular coordinates from eye position of points in the driver's environment; this field includes the vectors of angular motion around the driver's eye as he moves along the road. The acceleration field presents vectors of angular acceleration rather than velocity.

The problems considered in this part concern the use made by the driver of the positional, velocity, and acceleration fields. To affect driving, these characteristics have to be registered by the driver and the driver's sensitivity to them influences their utility. The analysis covers the condition of steady state driving, where the vehicle moves rectilinearly with constant velocity.

Equations of Position and Motion

The coordinate system is illustrated in figure 1. The driver's eye is at the origin and the road is considered to be on an infinite plane at some distance in z below his eye. Distance ahead of the eye is represented by x, to the side by y. Distance from the eye to any point of the field is represented by ρ, whose projection on the xy plane is r.

The equations for ground position and motion were derived previously. Equations (1) through (8) describe angular position velocity and acceleration under rectilinear, uniform, vehicular motion. These equations apply to the induced motion of the ground, and all other environmental points, viewed from a moving vehicle. The analysis of ground motion into separate azimuth ($d\theta/dt$) and declination ($d\phi/dt$) components seemed more appropriate than the development of an equation for total angular velocity (*1*, *2*). In some situations, the driver reacts differently to these components of motion (*9* .

Positional Field and Vehicular Guidance

The angular coordinates (θ, ϕ) of the ground plane from 0 to 50 feet to the left, and from 0 to 100 feet in front of the driver are shown in figure 8. As the driver sits on the left of the vehicle, the window areas and road views are asymmetrical. Although the left side is shown in figure 8, the flow lines also apply to the right side of the vehicle, if the appropriate window area is superimposed, and to the rear areas as well. The driver's eye is placed at a representative height of 4 feet above the ground. The shaded area at the right of the figure is the automobile cab and hood, which partially cuts off the view of the road. The blind areas at 0.65 and 0.9 radians are the roof supports.

The equirectangular projection shown in figure 8 is one of many possible ways of representing a three-dimensional environment on a flat surface. The figure is distorted; at the zenith and nadir of actual space, 360° of azimuth are reduced to a point, whereas in the figure they occupy the same extent as on the horizontal meridian. A rectification in the θ dimension may be achieved by curving the page through 90° and viewing it from a point close to the center of curvature. The ϕ dimension is not rectified, but it need not be as it covers a limited half-radian range.

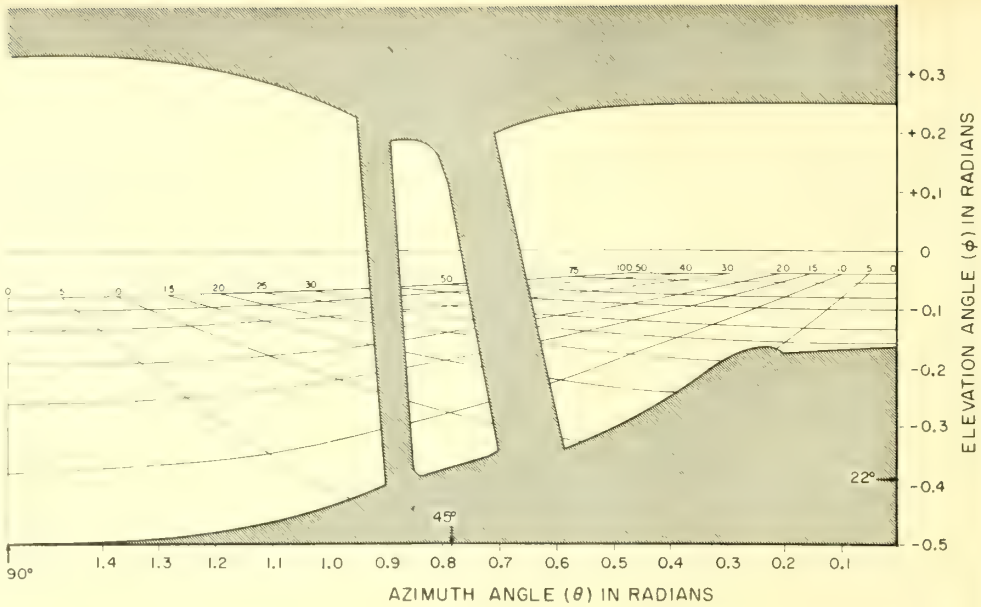

Figure 8.—Positional field through windshield and side windows of car.

Linear Perspective and Interpretive Scale

Linear perspective, the diminution of angular size with distance, is related to the positional field. The angular scale of the positional field may be expressed in terms such as $\Delta\theta/\Delta x$; in this instance it indicates the change in θ angle associated with a small change in x. If $\Delta \ell$ is a small change in x, y, and z, that is $\sqrt{\Delta x^2+\Delta y^2+\Delta z^2}$, and $\Delta\alpha$ is the change in angle associated with $\Delta \ell$, then angular scale is $\Delta\alpha/\Delta \ell$ in radians per foot or equivalent units. Thus, angular scale expresses the angular effects underlying perspective and establishes a relation between linear perspective and the positional field.

Interpretive scale may be defined as the inverse of the angular scaling effects underlying perspective. If angular scale is $\alpha/\Delta \ell$, then the corresponding expression of interpretive scale is $\Delta \ell/\Delta\alpha$ in feet per radians or equivalent units. Applied to a road map, interpretive scale represents the miles per inch required to interpret map lengths rather than the inches per mile used to draft the map. The perception of interpretive scale enables the driver to calibrate visual angle, in terms of length, on the road. Scaling may be explicitly in terms of feet and inches, but more commonly it involves distance exemplified by the driver's estimation of the space to a road point. The driver's conception of scale may be designated by primes ($\Delta \ell'/\Delta\alpha'$) to denote that a subjective estimate is implied.

As scaling involves the inverse of the angular effects underlying linear perspective, scaling would be expected to show up somewhere in space perception. Interpretive scaling is a key factor when the observer is making quantitative judgments of size, distance, or motion. In size perception, the observer estimates scale and then evaluates the visual extent of interest in this scale. Gibson describes the process as follows:

> ". . . with fixed monocular stimulation the size of an object is given by the size of the elements of texture or structure in the adjacent optical array. . . . Size is perceived relative to the size scale of the place where the object is seen." *(10)*

Scaling also applies to distance judgments. Gogel finds that judgments of the relative distance of objects can be explained in terms of the ratio, familiar size divided by retinal size *(11)*. This ratio is equivalent to interpretive scale. Gogel, referring to the evidence of other experiments, states that the ratio underlies the judged distance between objects of similar shape but different size *(3, 12, 13)*, different shapes *(14, 15)*, and familiar objects of different sizes and shapes *(11, 14)*. If the scale of one object appears smaller than that of another, it will be judged as being closer to the driver; if it is seen as equal, they will be judged as being equidistant from the driver; and if the scale of one object appears to be larger than another, it will be judged as being farther away from the driver. In the experiments discussed here, where background cues were minimized, judgments of relative distance were based upon relative scale.

Often distance judgments are made between widely separated objects. If the separation is large, the scale changes along the path and the judgment would be expected to take the following form, where the $\Delta_{\ell'}$ is a convenient length between the objects.

$$\text{Judged distance} \sum_{\ell'=1}^{\ell_1=n} (\text{scale of } \Delta_{\ell'})\Delta_{\ell'} \tag{44}$$

It may be predicted that the observer would summate in situations where the scale changes. A similar approach holds for judging distances from the eye, except that scale is learned through experience and does not have to be repeatedly resolved.

It seems reasonable to believe that interpretive scale, once resolved, would be applied to size, distance, and motion judgments in a situation. Thus, the scale of a familiar-sized object may be applied to distance and speed judgments in the same setting. Speed judgments may depend upon interpretive scale; the angular movement of objects seen by a stationary eye is converted to speed by the same rule that governs angle and length, but differentials are involved instead of time derivatives. For example the following equation would be used rather than equation (9).

$$\Delta\theta=\frac{-y}{r^2}\,\Delta x \tag{45}$$

Thus, Δx is converted into $\Delta\theta$ by the same function of y and r^2 as dx/dt is converted to $d\theta/dt$. The same approach holds for $\Delta\phi$. As angular stimulus is the vectorial sum of $\Delta\theta$ and $\Delta\phi$ components, and angular movement stimulus the sum of $d\theta/dt$ and $d\phi/dt$ components of motion, the communality of angle and angular velocity effects is demonstrated. The similarity of interpretive scales required for the correct perception of length and speed also is demonstrated.

The relationship between length and distance judgments is discussed in references under the size-distance invariance hypothesis *(16, 17, 18, 19)*. The hypothesis has been stated as, "A visual angle of given size determines a unique ratio of apparent size to apparent distance." *(20)* As the hypothesis describes a perceptual relationship, it must be shown valid on the basis of perceptual rather than geometrical evidence. Thus, if size and distance judgments are based upon the same interpretive scale, a rational basis would be provided for an invariance hypothesis. In some circumstances, the scaling underlying size and distance judgments differs. The scale in a film projection may be established by familiar objects included in the scene, which would not directly indicate the scale of eye distance. The same principle is true in size judgments of geological specimens. A hammer included in a page illustration may provide the size scale, but distance from the eye would not be shown thereby. The moon illusion is a puzzling example in which the seen size and distance are paradoxically unrelated (*21*). The relation of linear and velocity scales has received less research attention. Experiments by J. F. Brown may be interpreted as indicating that linear scale is actually applied in speed judgments (*22*).

The author finds it tempting to think that slant and shape perception could also be explained in terms of interpretive scale. Slant could be dealt with as the seen ratio of frontparallel scale to surface scale perpendicular to this line. If the ratio of these scales were one-half, the surface would be 60° to the line of sight, that is, cosine 60° is one-half. This explanation of slant perception is objectionable because it seems more complex than the judgment explained. The author tends to believe that slant and the perception of shape are basic perceptions that precede rather than follow scaling judgments. However, if an observer participating in a shape constancy experiment makes estimates of the dimensions of simple rectangles or ellipses projected at a slant to the eye, the responses may be considered to involve relative scaling.

The question remains of how scale is obtained. In theory, a decoding could be simply achieved as every x and y point on the ground plane has a unique θ and ϕ representation. A mechanical robot could steer down a level road and could avoid obstacles if it were able to sense θ and ϕ and if the road were correctly coded for it. Such a two-dimensional approach has an appealing simplicity but the nature of the visual world is three-dimensional. Scale is clearly shown by familiar sized objects, particularly those having parallel sides. The quantity $\Delta \ell'$ is then the known length of the familiar object; $\Delta\alpha'$ is its seen angular extent. An example is provided by the space perceptions of the motorist. An obvious key to the terrain configuration is the shape of the road ahead. The roadway is of almost constant width, it converges rapidly in perspective, and it provides a ready scale for converting visual angle to linear extent, which may be applied to other objects. Four road configurations are shown in figure 9.

In part 4 of figure 9, the projection of the road boundaries shows Δz above $L_1 = L_2$. If the borders of the road are straight, the road itself has no vertical curvature. If the borders are concave, the road surface is concave. If the borders are convex, the road is also. On an uphill, the road may rise above eye height; downhill it will be below eye level, and the road may in fact be overlapped and hidden. Light and shadow, texture, occular adjustments, intersections of surfaces, and overlapping of contours may enhance these perceptions. The roadway is a particularly convenient scale since it is always available, has known characteristics, and provides continual feedback to the driver on the accuracy of his perceptions. However, other familiar objects such as vehicles, pedestrians, houses, fence posts, sidewalks, and crosswalks are usually in the field of view and may also be used to calibrate visual angle.

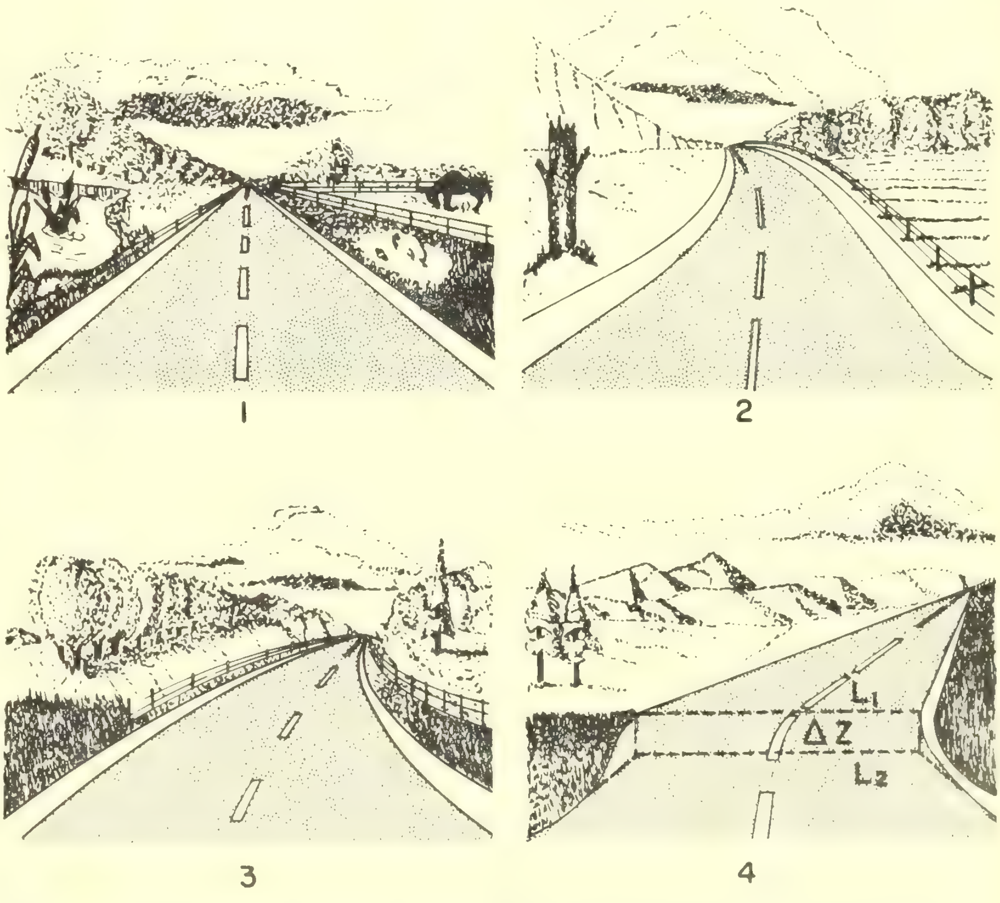

Figure 9.—Road configurations. Width of road provides linear scale; convergence pattern shows vertical curvature; in part 4, projection of road boundaries shows Δz above $L_1 = L_2$.

Angular Velocity Field and Vehicular Guidance

Velocity vectors along the flat ground plane have been plotted in figure 10. The ground area covered has been superimposed on figure 8; the magnitude and direction of the ground flow is shown by the length and direction of the vectors in the figure. The angular velocity field appears to fit almost exactly over the positional field. The resemblance is the result of the approximate equivalence of $d\theta/dx$ and $d\phi/dx$ vectors to $\Delta\theta/\Delta x$ and $\Delta\phi/\Delta x$ angular extents. As the eye moves in x, velocity vectors give the effect of equally sized rods parallel to the x axis. On the ground plane these vectors fall on perspective lines. As shown in figure 10, $d\theta/dx$ is zero along the median plane, approaches zero at $\theta=\pi/2$, and both $d\theta/dx$ and $d\phi/dx$ are zero at the vanishing point at eye level directly ahead. Objects off the path change their $d\phi/dx$ velocity component to predominating $d\theta/dx$ as they are approached.

Perception of motion

The psychological basis of motion perception is discussed in references (*4, 5*). At slow rates, motion is inferred from a change in position. A familiar example of this type of motion perception is the minute hand of a watch. Motion is interpreted by comparison of the positions assumed by the hand with the passing of time. At more rapid rates, motion is directly perceived. This perception is exemplified by the motion of the second hand of a watch, which actually is seen to move. At still more rapid rates, motion appears as a blur. Similar judgments and perceptions enable the driver to register impressions of motion related to the velocity field around him.

Basic reference for object motion

Just as the positional field provides a scale for object size, the velocity field might provide the background for the seen movement of objects. However, the driver may see the ground as a still reference or he may conceive that other cars are moving in reference to his own vehicle. For example, the driver reacts promptly when a vehicle approaching his moving car has no apparent sidewise velocity vector. This is visual warning of an impending collision. This relation was used by

Michaels and Cozan to explain vehicular avoidance reactions (9). An analogous situation occurs when the resultant vector is directed toward the median plane. The intruding vehicle will cross a driver's path and may collide. The driver uses his own vehicle as reference when he reacts to such situations.

The velocity field affects the driver's sensitivity to motion. The relatively small angular motions ahead of the vehicle favor the perception of moving objects, particularly in the θ direction. The fast movement of the field at $\theta=\pi/2$ inhibits perception of object motion. If the vehicle moves very rapidly, the resolution of independent motion is impaired by blurring. Angular velocities of several hundred degrees per second are generated at the side of a rapidly moving vehicle and the driver's vision is reduced even if he follows object movement with his head and eyes (*23, 24, 25*).

Perception of speed and direction of vehicular motion

The velocity field provides information on the speed and direction of the vehicle's forward motion. Although observed motion is the most direct indicator of vehicular motion, it is but one of its accompaniments. Vehicular speed is also indicated by direct speedometer readings, the pull of the steering wheel, the gear in use, the response to the accelerator, the pitch of the engine and tire squeal, the roughness of the ride, and centrifugal force when a curve is rounded. Apparently the visual appearance of motion cannot be claimed to be the only or even the most useful input for the estimation of speed.

The direction of vehicular motion is indicated by the flow characteristics of the velocity field. On a straight path there is no $d\theta/dt$ component in the median plane ahead of the driver. This lack of motion may be used by the driver, along with information of posture and the position of objects in the windshield, to indicate the direction of the vehicle's motion.

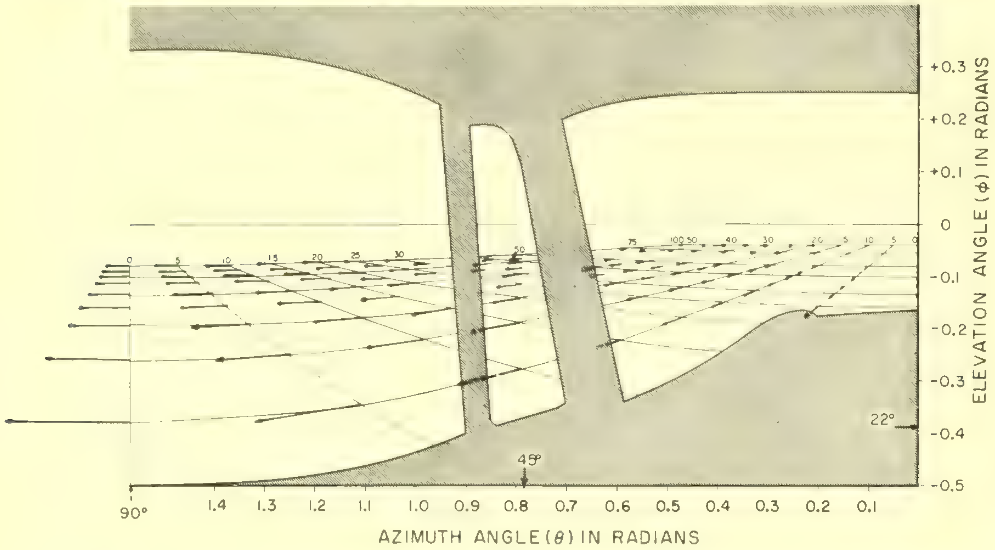

Figure 10.—Velocity vectors on basic ground plane. Direction and amplitude of angular velocity is indicated by direction and length of vectors.

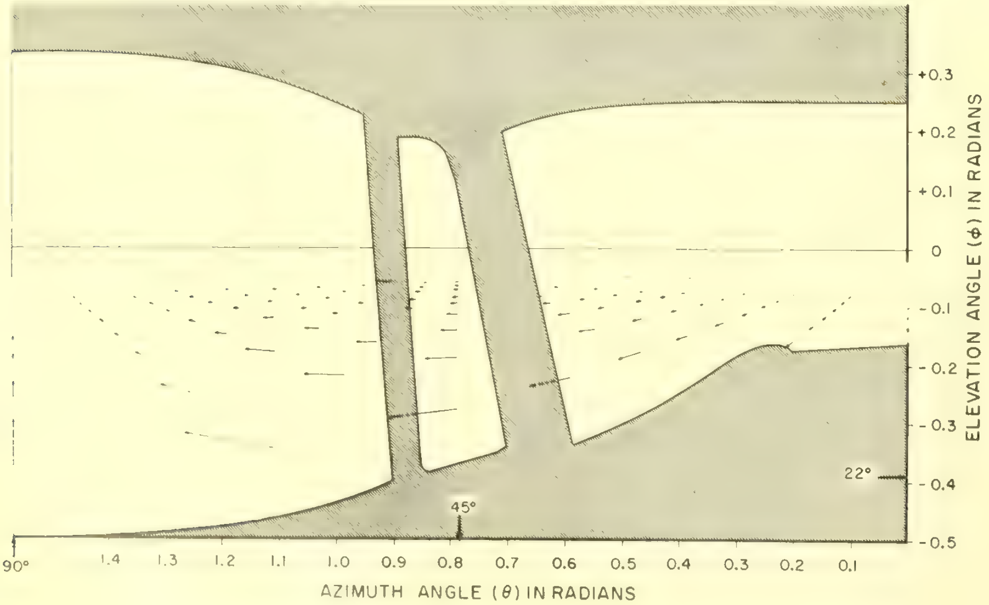

Figure 11.—Acceleration vectors on basic ground plane.

The importance of expansion patterns as an indication of the vehicle's direction of movement has been pointed out by James Gibson (*6*), as follows:

"When an observer approaches a surface instead of moving parallel to it, a modification of its deformation is introduced in that the focus of expansion is no longer on the horizon of that surface but at a particular spot on it—the point of collision with the surface. The rule is that all deformation in a forward visual field radiates from this point. Crudely speaking, the environmental scene expands as we move into it, and the focus of expansion provides us with a point of aim for our locomotion. An object in our line of travel, regarded as a patch of color, enlarges as we approach. It is not difficult to understand, therefore, why this expansion should be a stimulus for sensed locomotion as well as a stimulus for sensing the lay of the land. The behavior involved in steering an automobile, for instance, has usually been misunderstood. It is less a matter of aligning the car with the road than it is a matter of keeping the focus of expansion in the direction one must go."

Although the direction of vehicular motion is related to the focus of expansion, the focus itself is not an effective cue. The focus of expansion of a flat horizontal plane lies at the vanishing point in the sky or it will occupy points on trees or buildings if the road is curved. Generally, it is difficult, if not impossible, for the driver to locate the focus of expansion (figs. 5, 6, and 10) and, contrary to Gibson, the borders and lane markings are used in vehicular guidance. When the vehicle is off course, these lines have a lateral component of movement.

Acceleration Field

The projection of the acceleration field on the ground plane is shown in figure 11. The vectors on the field represent the differences in successive velocity vectors, divided by time, as time approaches zero. The cab area is the same as in figures 8 and 10, but the vector scale is 10 times as large. As shown in figure 11, there is no azimuthal component ahead of the eye under rectilinear motion, and $d^2\phi/dx^2$ is directed toward the eye in these positions. Vectors are directed away from the eye at $\theta=\pi/2$ where $d\theta/dx$ goes through a maximum. At angles between $\theta=0$ and $\theta=\pi/2$, the vectors shift from an approaching to a receding direction and are generally largest close to the eye.

Perceptual Problems

The major perceptual problem of the acceleration input is whether it is directly sensed. The human can distinguish accelerations, but it is not certain that they are detected as such. They may possibly be inferred from successive impressions of changing rates. Gottsdanker and Frick and Lockhard showed that group performance is more clearly ordered in terms of a threshold based upon total change in velocity than in terms of direct sensory impression of acceleration (*26*, *27*).

The vector field shown in figure 11 provides evidence on the sensing of acceleration. The field appears unnatural and there is no characteristic of experience that we can associate with the pattern of vectors shown. This differs from the psychophysical correspondence between light wave length and hue, physical energy, brightness, and so on. As the observer does not directly or precisely register accelerations, the acceleration field and the gradients within it cannot be considered a primary visual input. The same conclusion probably holds, by extension, to higher velocity derivatives. This is not to suggest that accelerations may not be perceived as changes in velocity.

Acceleration varies as the square of speed, as shown in equations (11) and (12). If speed of the moving eye is doubled, angular acceleration is quadrupled. The same condition holds for the eye on a curved trajectory. This relation leads to the paradoxical situation that the angular acceleration is more sensitive to speed than is angular velocity. It would be expected that the appearance of the environment would change markedly as linear speed is increased and that there would be acceleratory indications of velocity. The visual appearance of increased velocity on a roadway may be a sharp swoop of objects and road features as they change from a ϕ to a θ direction. The imperfections of lane markers and road edges may show acceleratory jitters. These acceleratory effects, however, have not been systematically verified.

Part 3—Motion Parallax and Perceptual Hypothesis Testing

Introduction

WHEN A PERSON is driving a car or walking, objects seem to pass on either side; distant objects seem to move more slowly than those close by. The relative motion of seen objects, formally called motion parallax, might be expected to provide an indication of distance. Discussions of depth perception, such as in introductory psychology textbooks, mention motion parallax as a classical cue to distance, along with visual angle, interposition, linear perspective, aerial perspective, and shadows (*28*, *29*, *30*, *31*).

Motion Parallax as Distance Indicator

The first thorough discussion of motion parallax as an indicator of distance is given by H. von Helmholtz in his volume of *Physiological Optics* (1866). According to translation, Helmholtz wrote (*5*):

"In walking along, the objects that are at rest by the wayside stay behind us; that is, they appear to glide past us in our field of view in the opposite direction to that in which we are advancing. More distant objects do the same way, only more slowly, while very remote bodies like the stars maintain their permanent positions in the field of view, provided the direction of the head and body keep in the same directions. Evidently, under these circumstances, the apparent angular velocities of objects in the field of view will be inversely proportional to their real distances away; and, consequently, safe conclusions can be drawn as to the real distance of the body from its apparent angular velocity.

"Moreover, in this case there is a relative displacement of objects at different distances with respect to each other. Those that are farther off as compared with those that are nearer seem to be advancing with the observer, whereas those that are near seem to be coming toward him; and the result is we have a very distinct apperception of the fact that they are unequally far from us. Suppose, for instance, that a person is standing still in a thick woods, where it is impossible for him to distinguish, except vaguely and roughly, in the mass of foliage and branches all around him what belongs to one tree and what to another, or how far apart the separate trees are, etc. But the moment he begins to move forward, everything disentangles itself, and immediately he gets an apperception of the material contents of the woods and their relations to each other in space, just as if he were looking at a good stereoscopic view of it."

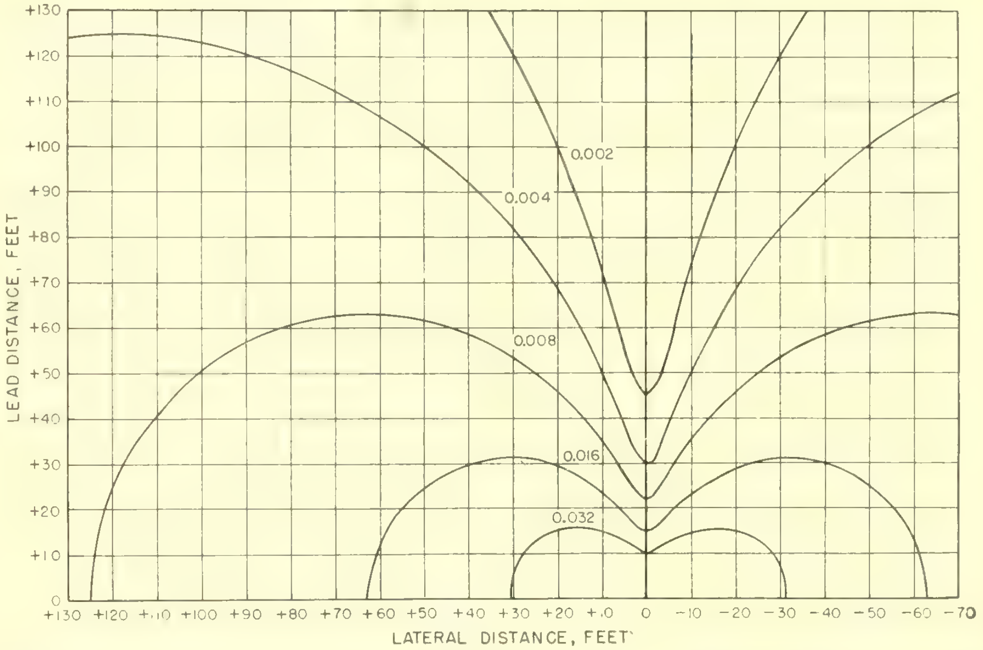

Figure 12.—Isoangular-velocity curves under linear motion. Angular velocity in radians per second of each contour can be obtained by multiplying value shown for each curve by vehicular speed in feet per second.

Helmholtz's statement that ". . . safe conclusions can be drawn as to the real distance of the body from its apparent angular velocity", can be questioned. When the observer follows a curved path, angular motion of ground points becomes an unreliable indicator of distance. The basic geometry of the situation is altered, so that motion of the ground or other points does not decrease regularly along a sight line. This is shown by the velocity field of curvilinear motion illustrated in figure 5 and by a comparison of isoangular-velocity curves of linear and curvilinear observer motions shown in figures 12 and 13. The curves are based upon equations (9), (10), (19), and (20) derived for object movement under linear and curvilinear observer movements. Isoangular-velocity curves involve a fixed sum of azimuthal and elevation velocity components, which add vectorially as

$$\sqrt{\left(\frac{d\theta}{dt}\right)^2+\left(\frac{d\phi}{dt}\right)^2} \qquad (46)$$

An infinite number of specific combinations of $d\theta/dt$ and $d\phi/dt$ may produce a given sum. Two combinations making up the sum 0.001 are $d\theta/dt=0.000707$ $d\phi/dt=0.000707$, and $d\theta/dt=0.0008$ $d\phi/dt=0.0006$. Each combination when substituted in equations (9) and (10) or (19) and (20) can be solved for x and y and hence yield a point for plotting on an isoangular-velocity curve. In practice, the solving of these equations was done on an electronic computer.

Complications

The vector field of horizontally curved observer motion depicted in figure 5 shows a number of features that complicate a motion

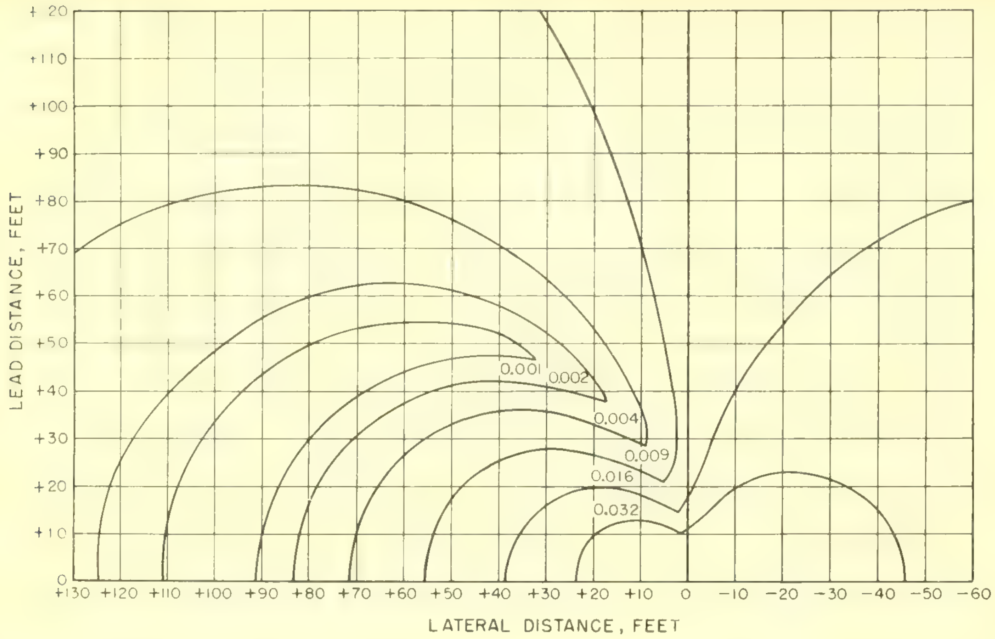

Figure 13.—Isoangular-velocity curves under curvilinear motion. Center of curvature is 100 feet to left of origin. Angular velocity in radians per second for each contour can be obtained by multiplying value shown for each curve by vehicular speed in feet per second.

parallax interpretation. As distance from the eye increases along an azimuth line, the direction of ground motion changes as well as magnitude. Motion on 0.3 radian azimuth line is to the right at far distances and to the left at close points. The interpretation of distance from these motions alone would be difficult. A somewhat similar objection has been previously raised by Gibson who considered linear motion and noted that angular velocity relates to distance only along sight lines off the vanishing point directly ahead (*1*).

Difficulties in using motion-parallax under curved motion are shown in the isoangular-velocity plots. These curves show the locus of terrain points of equal angular velocity, regardless of direction of motion. Thus, linear isoangular-velocity curves decrease fairly systematically with the reciprocal of distance on each azimuth. The curvilinear isoangular-velocity pattern is different. The functions are asymmetrical and approach a limiting value of $1/R\ dx/dt$, where R is the radius of curvature and dx/dt the speed of the motion. Angular movement, as shown in figure 14, is seen to reverse itself on the sight line through the center of rotation at $x=0$, $y=100$ feet. An interpretation in accord with the motion parallax approach would place the stationary center of rotation at infinite distance from the eye. Positions beyond the center of rotation on the same azimuth line increase in angular velocity and hence would be interpreted as decreasing in distance. Evidently, distance is not related to ground motion in the manner proposed by Helmholtz, when the vehicle is following a curved path.

On a straight trajectory, distances at right angles to the line of movement where angular velocity is high are not noticeably easier to estimate than those ahead where angular velocity is minimal. On the contrary, illusory movements of the terrain are seen to the side, which depend upon the observer's visual fixation position. If the foreground is viewed from a car, the background seems to move and rotate forward around it. If the background is fixated, the foreground seems to turn. This illusion of rotation, which may be called motion parallax curl, is based on differences in angular velocity between the foreground and background as shown in figure 14. The point of reference of seen movement is ambiguous and is not necessarily the vehicle itself, as a motion parallax formulation would require. Von Kries mentions this illusion: He believes that the reference point of motion is the stationary point of fixation, according to a note on page 372 of reference *5*. The magnitude of the illusion is given by the following derived formula:

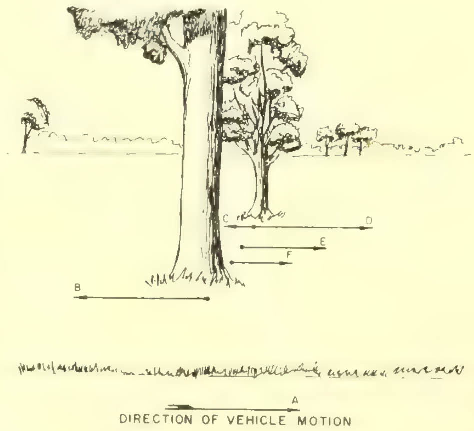

Figure 14.—Motion parallax curl. Illusion shown at right angles to vehicle's line of movement. Vehicle moving in direction A, inducing vectors B in foreground tree and vectors C in background tree. If foreground tree is fixated, background moves to right with velocity D. Ground positions between trees have linearly decreasing velocity vectors that produce appearance of rotation.

$$V=\left[\frac{(r_1-r_2)}{r_1r_2}\sin\theta\right]\frac{dx}{dt} \qquad (47)$$

Where,

V = angular velocity of curl, rad./sec.
θ = angle of object from line of motion
r_1 = distance of foreground fixation
r_2 = distance of background object
dx/dt = vehicle's speed, ft./sec.

The illusion increases with the distance between the foreground and background, velocity of the vehicle, and lateral angle. It decreases with remoteness of both foreground and background points from the eye. A parallax curl is also seen when tree limbs, telephone wires, and other objects above the horizontal are viewed. However, this form of the illusion seldom is noticed.

Experimental evidence lends support to the geometrical analysis. The context of motion must be revealed to the observer if he is to see depth—differential motion alone is not sufficient to produce the experience (*32*). Thus, the movement of a pattern produced by a shadow caster gives the appearance of varying depth only if the pattern is seen as a surface. If it is seen as a moving surface, a more accurate judgment of slant is given than if the surface is stationary (*33, 34*). If the observer surmises that he is viewing objects at different distances, as when he sees luminous patches in a dark room, he can estimate relative but not absolute object distance by moving his head (*34, 35*). If the arrangement of a shadow caster is described to an observer, he can make relatively accurate depth judgments of moving objects even though movement itself may not be experienced (*34*). These results show that it is difficult, if not impossible, to estimate distance from object movement alone.

Perceptual Hypothesis Testing

Examples exist in the literature, which indicate that motion does facilitate space perception. The effect is not through motion parallax, but by what may be called perceptual hypothesis testing. Other facilitative effects of motion are the kinetic depth effect (*36*) and minute image movements (*37*). These phenomena are not closely related to motion parallax so are not considered herein. When the observer alters his viewing position, perspective relationships change in a unique manner dictated by the structure of the field and the movement made. This change permits previous hypotheses concerning the functional nature of objects and surfaces to be verified or amended. Perceptual hypothesis testing is also related to the Transactional

Theory of Perception (*38, 39*) and to Brunswick's Probabilistic Functionalism (*40*). As shown by Ames (*38*), Gibson (*6*), Gogel (*41*), and others, a static view may lead to multiple or even illusory perceptions. But the faulty perception will not pass the test of redundant viewing under movement; it is not supported by the perspective changes that occur. Hypothesis testing, involving observer motion, also may involve testing by coherency. This principle is illustrated by Helmholtz's tree example. Under observer movement, a single tree among others shows a coherent movement that differs in speed and extent from that of other trees differently placed. The induced movements serve to resolve the scene into spatially separate components. This simple coherency principle aids space perception, regardless of the observer's path of motion or fixation position. Coherency testing is related to Wertheimer's "law of common fate," according to which points moving simultaneously in the same direction are readily seen as a group (*42*).

In certain limited and special situations, observers have been reported to use motion parallax information to tell distance. Such cases include among others the animal studies by Wallace, Pumphrey and Grinnell, and Gibson and Walk, which have been summarized by Shinkman (*43*). Experiments involving infants placed on an optical cliff have also been interpreted to indicate the importance of motion parallax (*44*). The author questions whether these situations really involved the deduction of distance from angular movement of two or more separated objects. Much of what has been interpreted as the operation of motion parallax may perhaps be explained as hypothesis testing involving change of perspective.

Part 4—Perceptual Mechanisms in Vehicular Guidance

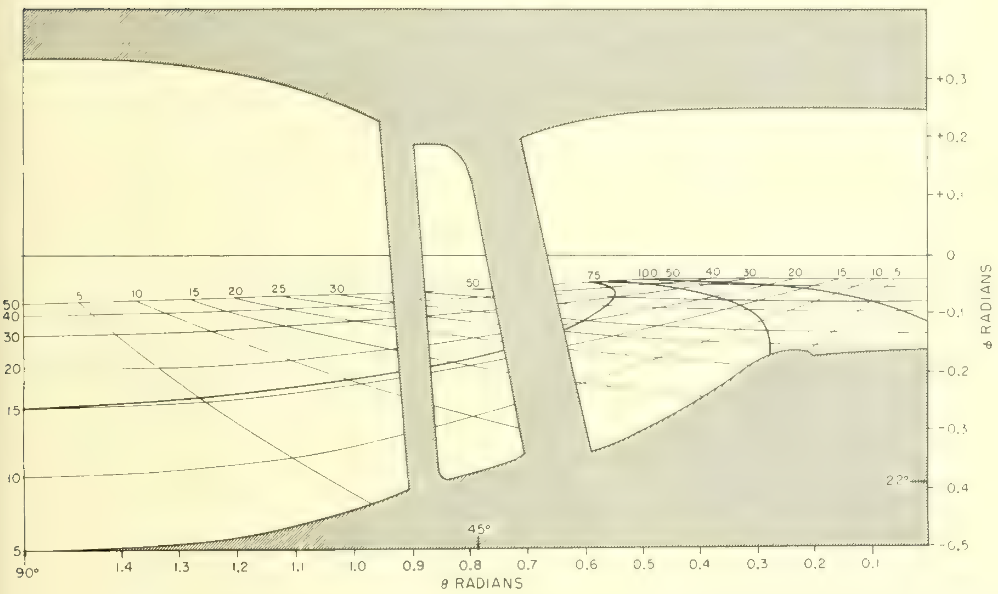

Figure 15.—Positional field through windshield and side window on a curved road. Left and right road borders and center lane divider are shown.

Introduction

IN THIS PART of the article curvilinear vehicular motion is discussed in addition to rectilinear motion, and the results are applied to the basic maneuvers of lateral guidance (steering), anticipation, and car following.

It is important that the curvilinear motion of the vehicle be studied. Visual studies of aircraft landing (*45, 46, 2*), car following (*47, 48*), and spatial orientation in motion have been exclusively on rectilinear motion. The aircraft or automobile often follows a curved path and, even on a supposedly straight path, the actual path of the vehicle will be slightly curved. Rectilinear motion may, in fact, be considered as a special occurrence of curvilinear motion, where the radius of curvature becomes infinite. Several accepted guidance theories, plausible enough for application to driving on straight roads, fail to apply generally to the curvilinear situation.

Position and Motion

The derivation of angular position and the motion of ground points under curvilinear movement of the eye are discussed in the first part of this article. The coordinate system is shown in figures 1 and 2. In these figures, the driver's eye is placed at the coordinate origin, distance ahead is represented by x, to the side by y, and up and down by z. Distance from the eye to any point of the field is represented by ρ, whose projection on the xy plane is r. Equations (1), (2), (5), (6), (7), and (8) hold for horizontally curved, constant speed, vehicular motion. On curves to the right the equations are:

$$\frac{d\theta}{dt}=\left(\frac{-y}{r^2}-\frac{1}{R}\right)\frac{dx}{dt}\text{ rad./sec.}\qquad(48)$$

$$\frac{d\phi}{dt}=\frac{xz}{r\rho^2}\frac{dx}{dt}\text{ rad./sec.}\qquad(49)$$

$$\frac{d^2\theta}{dt^2}=\left(\frac{2xy}{r^4}+\frac{x}{Rr^2}\right)\left(\frac{dx}{dt}\right)^2\text{ rad./sec./sec.}\qquad(50)$$

$$\frac{d^2\phi}{dt^2}=\left[\frac{-z}{\rho^4r^3}(\rho^2y^2-2r^2x^2)-\frac{zy}{\rho^2rR}\right]\left(\frac{dx}{dt}\right)^2\text{ rad./sec./sec.}\qquad(51)$$

The positional equations (1) and (2) are identical to those for linear translation, thus indicating that perspective is unaffected by the path of motion. The declination angular velocity, equation (6), also is unchanged by circular motion. But azimuthal angular velocity, $d\theta/dt=\left[(-y/r^2)+\frac{1}{R}\right]\frac{dx}{dt}$, differs from the linear, $d\theta/dt=-y/r^2\,dx/dt$ by the constant $1/R\,dx/dt$, which is equal to angular velocity, $d\beta/dt$. The velocity effects of circular field movement therefore involve the simple addition of translational and rotational effects. The expressions for $d\theta/dt$ and $d^2\theta/dt^2$ are independent of viewing height, z. In contrast, expressions for $d\phi/dt$ and $d^2\phi/dt^2$ do involve viewing height. These equations of position and motion have broad applicability. They govern the angular position, velocity, and acceleration of any point in x, y, and z. These equations also may be modified to describe the seen movement of vehicles, pedestrians, and other objects that are in motion.

Positional Field

The positional field is illustrated in figure 15. The driver's eye is placed at a representative height of 4 feet above the ground. The vehicle is shown on a sharp curve of 100-foot radius. As shown by figure 15 a road of constant curvature assumes a steady state appearance up to the break in curvature. The existence of a steady state is important in describing driving. Lateral guidance and car following could consequently be considered to involve the maintainance, through visual feedback, of an acceptable steady state condition and the nulling of deviations from it.

The positional field shown in figure 15 is a graphing of a mathematical relationship; yet it is readily interpreted as the view from inside a car. Perspective lines on the positional field provide sufficient information to elicit a perception of the ground plane, even

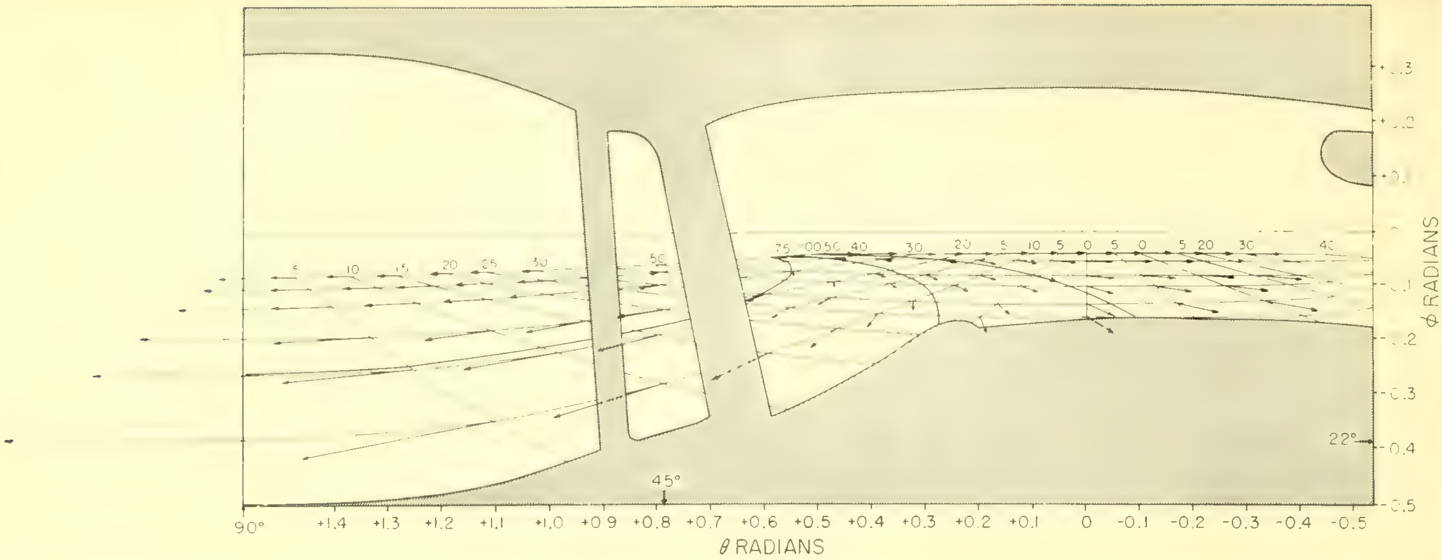

Figure 16.—Velocity vectors on basic ground plane. Direction and amplitude of angular velocity is indicated by direction and length of vectors.

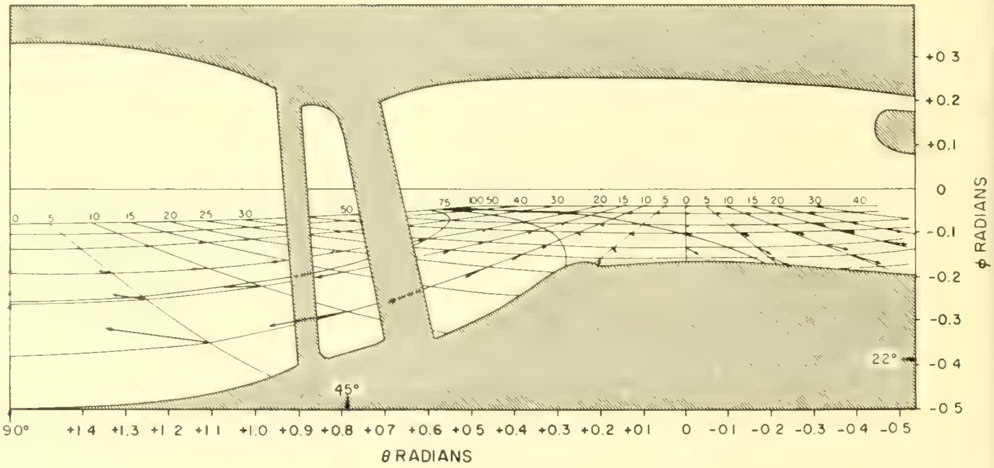

Figure 17.—Acceleration vectors on basic ground plane. Direction and amplitude of angular acceleration is indicated by direction and length of vectors.

though texture, binocular disparity, road objects, motion, and other information are absent. Previously, it was suggested that the interpretive scaling of visual angle is a fundamental process in length, distance, and motion estimation, and that the configuration of the road might enable the driver to obtain this scale. Interpretive scaling also applies in curvilinear motion, as the positional field is not affected by the vehicle's path of motion.

Velocity Field

The velocity field on the basic ground plane is plotted in figure 16. The magnitude and direction of the ground flow is shown by the length and direction of the vectors. If the vehicle accelerates or decelerates, the vector pattern remains unchanged, but individual vectors will change proportionally in magnitude.

The curvilinear velocity field resembles the linear field to the side of the vehicle where the vectorial effects of simple translation are themselves large. The field differs at distant points, where rotational effects overshadow those of translation. The field is asymmetrical; angular rotation is added to azimuthal velocity on the side opposite the center of curvature and subtracted from the near side. The velocity vectors are tangent to the road boundaries and lane markings, a condition necessary for the existence of a steady state. The only stationary point on the field is the center of circular curvature located R feet to the side of the vehicle.

The velocity field is perceived as a positional field in motion. The moving observer does not see a confusing field of independent motions, such as might be visualized by examining separate points of figure 16. Rather, the velocity vectors are perceptually organized, or inherent, in a unitary conception of a terrain in motion. The velocity vectors are a function of the speed and direction of the observer's motion, and unlike the visual angle stimulus, do not reveal the scale of object length.

A velocity projection such as shown in figure 16 is generated by an infinite number of stimulus conditions. If distance to the surface is multiplied by a constant, and speed as well, the vectorial pattern is unchanged. Distance to the surface must be specified to permit a judgment to be made of the speed of motion. The perception of movement implies a perceived relationship between the eye and moving environment. If references are absent, as in a film record of a featureless terrain of sand or brush, the direction of motion is difficult to determine. Experimental evidence suggests that the driver guides himself by reference to the road edges and the center stripe (*49*).

Acceleration Field

The angular acceleration field is presented in vector form in figure 17. The field represents differences in successive velocity vectors, divided by time, as time approaches zero. The cab area and ground of figure 17 are the same as in figure 16, but the vector scale is 10 times as large. The direction of the field vectors resembles the direction of perspective flow lines at large θ angles, but the direction is otherwise rather irregular. The field differs from the acceleration field of linear motion, particularly at $\theta=0$, where the vectors are directed toward negative θ rather than straight toward the observer.

The acceleration field associated with curvilinear vehicular motion relates to problems previously considered in relation to rectilinear motion. Analogously, the peculiarities of the acceleration field pattern, particularly at $\theta=\pi/2$ provide evidence that angular acceleration is not directly sensed. The equations for curvilinear acceleration also contain a $(dx/dt)^2$ term, indicating the sensitivity of acceleration to speed and raising the expectation that there would be accelerative indications of velocity.

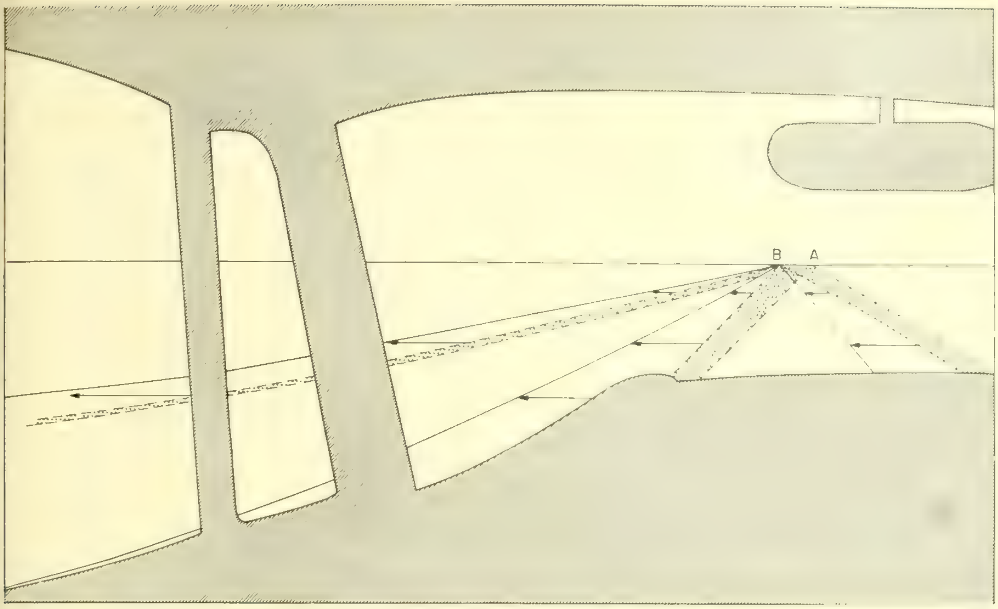

Figure 18.—Lateral guidance on a straight road. Initial slewing is shown by shaded area and sideslip by arrows.

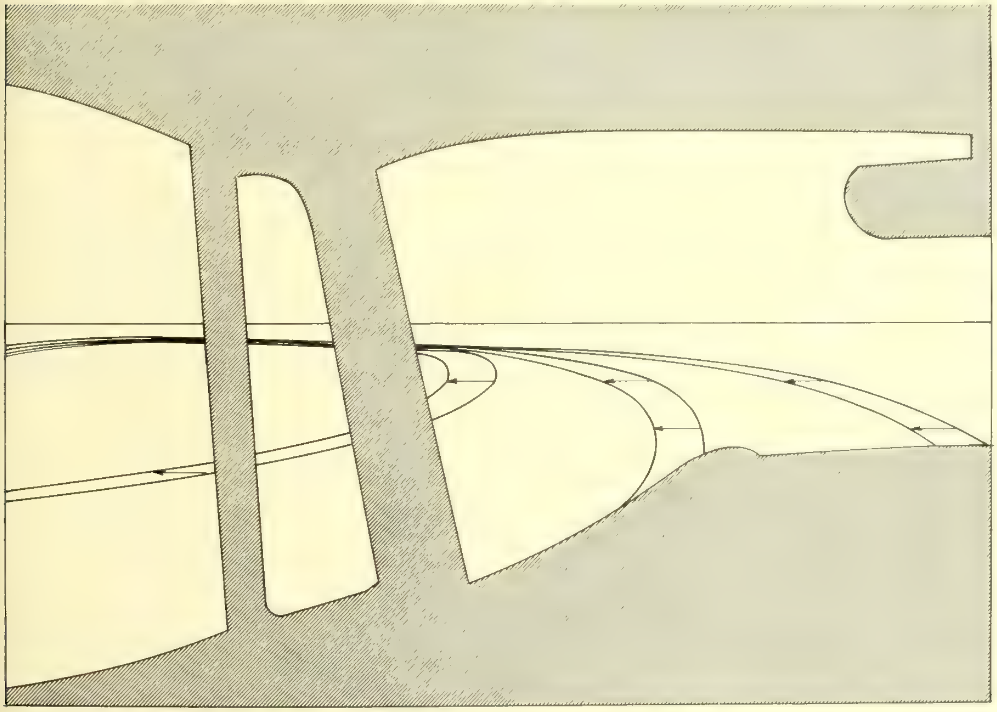

Figure 19.—Lateral guidance on curved road. Sideslip is shown by arrows.

Vehicular Guidance Modes

The previous discussion now may be applied to explain the vehicular maneuvers of lateral guidance (steering), perceptual anticipation, and car following. These guidance maneuvers have been the subject of other research and are accepted as important driving activities (*47, 48, 50, 51, 52, 53*).

Steering

When the moving vehicle is alined with the highway, each point on the road border and lane marker falls on the angular position previously occupied by another point of the border, and the road assumes a steady state appearance. When the steering wheel is turned, the road borders and lane marker appear to move in the opposite direction. All parts move; no one part is essential for tracking. In fact, the driver may assume a somewhat unlocalized surveillance of the road, which would facilitate seeing a steady state and also reduce the involuntary rapid movement of the eyeball, which is technically referred to as nystagmus. The driver may become aware of the misalinement of the car by slewing shifts in direction, and by sideslipping, sidewise movements. These effects are illustrated by the perspective diagrams of figures 18 and 19. A 2½° shift in angle on a straight road is shown in figure 18. The initial 2½° slew exceeds the human visual acuity threshold, which is about a minute of visual angle (*5*). If the movement is executed in less than a second, the movement also exceeds the threshold for visual motion, which is about 2 minutes of arc per second (*5, 54, 55*). Sidewise movement to the road borders is perceptible either as a movement or change in position.

The visual effects of misalinement on a curved road are illustrated in figure 19. At 30 m.p.h. velocity, the tire will hit the shoulder of the 100-foot radius road in 1½ seconds. Average slewing rate is 10° per second over the 15° shift in angle. Evidently, amount or rate of slewing, or amount or rate of sideslipping, may provide the driver with feedback from the maneuver. Even if road movement is as small as 3½ inches (1/12 the values of the illustration) sideslipping and slewing on a straight road would exceed human visual position and movement thresholds. On the basis of human perception theory, it is difficult to determine which of the four combinations of slew, sideslip, rate, and amplitude the driver perceives. The driver responds to a total situation, not to isolated or ranked cues. If the vehicle changed heading but did not sideslip, the driver would rightly conclude that it was slipping on ice. Other visual inputs would confirm the impression. If the road moved sidewise without a change in heading, the driver would conclude that his vehicle was entering a curve. The perceived situation is a hypothesis that best reconciles all inputs.

Questioned

Several theorists have emphasized local characteristics of the velocity field as aids in lateral guidance. Gibson claims that the center of expansion is the prime cue used by the driver to determine his direction of movement, as quoted in part 2 (*6*). A difficulty with this theory is the indiscernibility of the center of expansion against the sky background. Additional difficulties are posed by curvilinear motion. On a curved trajectory, the entire field moves and the center of expansion vanishes, as shown in figure 16. The only still point on the field is the center of rotation that lies far to the side of the driver's line of regard.

A similar difficulty applies to the theory that the null $d\theta/dt$ locus may serve as an indicator of the vehicle's path of motion. The null $d\theta/dt$ locus is made up of all the vectors aimed directly toward the vehicle. This locus coincides with the future path when the car moves rectilinearly as shown in figure 20. In the curvilinear situation, the pattern becomes a semicircle between the eye and the center of rotation that does not coincide with the vehicle's path. Another cue, based on the velocity field, that motion parallax is an indication of depth, also fails when the vehicle follows a curved path. Evidently, the features of the velocity field are path dependent and are too unstable to provide guidance alinement on curved paths.

Perceptual Anticipation

Perceptual anticipation is of central importance to the driving task. The design of highways must permit the driver to anticipate ahead. If the driver's view is limited by fog, rain, or lack of illumination, he slows down or stops; and if forced to maintain speed, he feels uncomfortable. Perceptual anticipation is illustrated in figure 21 by Taragin's data on driver behavior on curved roads (*56*).

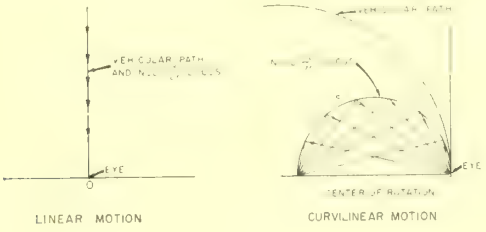

Figure 20.—The null dθ/dt locus in linear and curvilinear motion.

He reported that most drivers adjust their speed on the approach to a curve and do not change it appreciably after entering the curve, as illustrated in figure 21. Operating speed was related closely to the degree of curvature and only slightly to sight distance on the curve. Anticipation effects also have been shown in the experimental data on lateral guidance (*9*) and in studies of driver behavior on vertical curves (*57*).

The driver must anticipate at least one reaction time ahead if he is to meet the current situation. Cumming (*50*) states this relationship as:

"In a tracking task . . . the reaction time is about 0.3 seconds; i.e., there is a time lag of 0.3 seconds between a signal and the corresponding action. This leads to the strategy, in the development of a tracking skill, of taking a preview of the required path and programing ahead to compensate for reaction time . . . . As the skill develops it becomes possible to extract the 'constants' to allow for smoothness and coordination and so be able to program further ahead than half a second.

"It is this ability to view the process in an unhurried way on broader time scale, with longer ballistic actions between instructions and without monitoring, that signifies a 'skill' in the generally understood sense."

The driver must also provide time for shifting his mental gears, for adjusting his planning, and for coping with emergency highway conditions that might arise. Warning signs and signals should be placed well in advance of the hazardous situation they refer to. The driver's visual fixation distance has been related to anticipation requirements. Wohl (*53*) believes that fixation position may be predicted by the following equation.

$$D = \tau V \qquad (52)$$

Where,

D = Sight distance ahead
τ = Driver's response time lag
V = Vehicle velocity

The equation implies that the driver looks ahead a distance equal to vehicular velocity multiplied by human response time lag. If the driver did not look at least this far ahead, he could not respond appropriately. However, even casual observation of driving behavior indicates that the operator does not view a fixed distance ahead. Rather, he looks far ahead, returns to middle distance, and, seemingly in disregard of anticipation requirements, he may check his alinement with the road and nearby vehicles. His behavior is more variable than the equation would demand. It seems easier to accept the following rearrangement of Wohl's equation.

$$\tau_A = \frac{D}{V} \qquad (53)$$

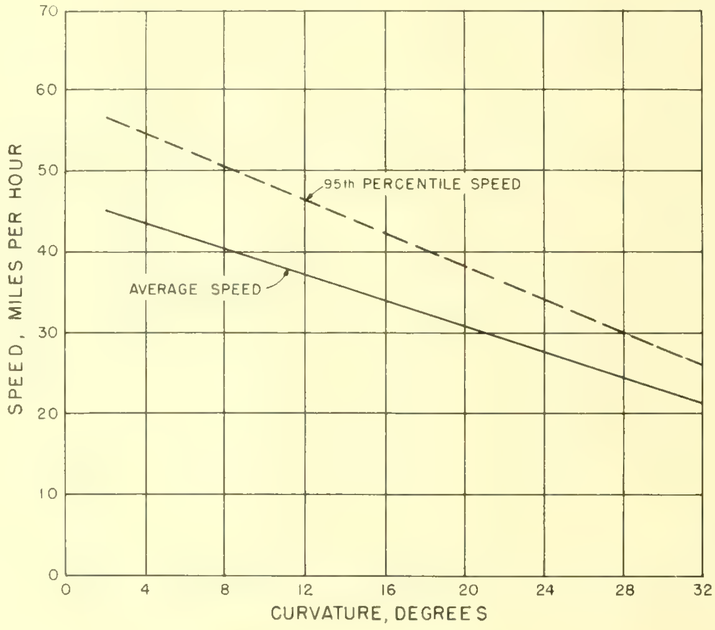

Figure 21.—Relation between speed and horizontal curvature. Speed on the curve decreases as the sharpness of curvature increases.

If the vehicle's velocity is fairly uniform, seen distance to a point ahead is linearly convertible to anticipation time. It is unfortunate that so little is known about the important factor of anticipation in driving. Exploratory studies are needed to determine optimal or minimal anticipation distances applicable to road signs, barriers, curves, and so on. The studies should be carried out at different vehicular speeds, under different visibility conditions, and with different car dynamics.

Car Following

In traffic, the driver's acceleratory and braking responses are made primarily in response to movements of the vehicle ahead. Car following responses may therefore be the basic element of complex traffic states, such as highway shock waves and instabilities causing accidents (*47*).

Car following has been intensively studied by Herman and his coworkers (*47*). On the basis of experimental observation on a test track and in New York City tunnels, the following equation has been derived for the car following process

$$(d^2x/dt^2)_{n+1} = \alpha_0 \frac{(dx/dt)_n - (dx/dt)_{n+1}}{x_n - x_{n+1}} \qquad (54)$$

Where,

$(d^2x/dt^2)_{n+1}$ is the acceleration of the following car.
$\alpha_{,}\pi 8_0$ is a constant related to speed.
$(dx/dt)_n - (dx/dt)_{n+1}$ is the difference in speed between cars.
$x_n - x_{n+1}$ is the headway distance between cars.

Herman instructed his subjects to ". . . follow at a minimum safe distance." These instructions do not include all possible driver intentions but they may cover the important situations of driving in single lanes, on bridges, and in tunnels. A perceptual basis for Herman's equation has been suggested by Michaels (*51*), in terms of the driver's seeing a change in the angular subtense of the vehicle ahead. If the following driver overtakes from a long distance, he can detect absolute changes in size of the vehicle ahead, but not the actual motion of the lead vehicle. When separation distances become sufficiently short, he will be able to detect angular velocity by expansion of the lead car's horizontal angle, whose rate of change is as follows.

$$\frac{d\theta}{dt}=K\,\frac{(dx/dt)_{n}-(dx/dt)_{n+1}}{(x_{n}-x_{n+1})^{2}} \qquad (55)$$

Where,

$d\theta/dt$ is the angular velocity of expansion of the lead car, width in rad./sec.

K is the width of lead car, feet.

A close relation is shown in the form of Herman's equation and the angular expansion equation, which suggests that Michaels has provided a perceptual basis for car following.

Michaels' explanation implies a close relation between the driver's perception and his response. Such automatic reaction may be an adequate model for Herman's experimental situation, which involved close monitoring of speed, but it is unlikely that the driver in traffic gives immediate response to his perceptions. Field observations by Perchonik and Seguin (*58*) indicated that the factor of headway distance was effective only at short vehicle spacing and that relative speed exerted maximum influence for vehicle separations in the 50- to 100-foot range. The factors in Herman's equations had no significant effects at longer headways (*49*).

As the driver is observed, he is not continually adjusting his velocity relative to the car ahead. Rather, he avoids coming dangerously close to that car and also avoids drifting so far behind that the gap will be filled by a car in an adjacent lane. If this approach is valid, then speed, traffic density, and the position of cars in adjacent lanes would affect car following, as well as the relative speed and position suggested by Herman. These opposing suggestions as to the basis of car following should receive a field test evaluation.

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Updated versions of two publications providing information on Federal Aid for Highways have recently been issued by the Bureau of Public Roads, U.S. Department of Commerce. *America's Lifelines—Federal Aid for Highways* and *Federal Laws, Regulations, and Other Material Relating to Highways* may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. A brief description of each publication and its purchase price is given in the following paragraphs.

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VOL. 34, NO. 4

OCTOBER 1966

Public Roads

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Types of some of the vehicles tested in the research reported in this magazine on *Braking Performance of Motor Vehicles* and the *Relation of Gross Weights and Horsepowers of Commercial Vehicles.* The third article *Offtracking Calculations for Trailer Combinations*, refers to some of the vehicle types shown.

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Relations of Gross Weights and Horsepowers of Commercial Vehicles, *by J. M. Wright and S. C. Tignor* 84

Offtracking Calculations for Trailer Combinations, *by Hoy Stevens, S. C. Tignor, and J. F. LoJacono* 89

NATIONAL BUREAU OF STANDARDS DEDICATION

Dedication ceremonies for new facilities of the National Bureau of Standards, Gaithersburg, Md., November 15 will be presided over by John T. Connor, Secretary of Commerce, and Dr. Allen V. Astin, Director, National Bureau of Standards. Dignitaries from Government, science, and industry also will participate in the ceremonies. To celebrate the construction and dedication of this new standards and testing complex, Secretary Connor is sponsoring an International Symposium of Technology and World Trade on November 16 and 17 at Gaithersburg.

The new $120 million NBS laboratory complex consists of 15 major buildings constructed on a 565-acre site. New laboratories and supporting facilities enable the Bureau of Standards to update its research programs in a rural environment removed from urban mechanical, electrical, and atmospheric disturbances. Expanded facilities include a nuclear research reactor and a linear electron accelerator for the establishment of measurements and standards. An Engineering Mechanics Laboratory was also added to the relocated facilities for work such as the calibration of rocket thrust measuring devices.

At the International Symposium of Technology and World Trade on November 16 and 17, experts from all over the world will examine and forecast the impact of technology upon the patterns and conduct of international trade and investment; consider the international environment needed for the wider generation and utilization of technology; and explore prospects for evolving policies and institutions that promote economic developemnt through technology and trade.

U.S. DEPARTMENT OF COMMERCE
JOHN T. CONNOR, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

Public Roads

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Braking Performance of Motor Vehicles

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

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Introduction

BECAUSE IT is well known that adequate brake performance is necessary for the safe operation of motor vehicles, the Bureau of Public Roads in 1941 undertook a research program to determine at periodic intervals the brake performance levels of motor vehicles operating on the highway systems of the United States. Studies also were made in 1949 and 1955. The most recent series of tests was made beginning in July 1963; results of these tests are discussed in this article. The 1963 field testing was done as nearly as possible in the same locations in Maryland, Michigan, and California used for the 1949 and 1955 tests. The information obtained from this series of braking studies is expected to be used to:

- Promote improvement in the general level of brake performance.
- Serve as a basis for revising brake performance standards.
- Provide current motor-vehicle brake performance data that can be used to establish highway design standards, such as standards for stopping sight distance.
- Show the levels of brake performance for the different types of vehicles using the highways.

Since 1941, the Bureau of Public Roads periodically has conducted a research program to determine the braking performance levels of motor vehicles operating on public highways. The research results are used to promote improvement in the general level of brake performance for all types of vehicles, provide information that may be used in establishing highway design standards, and serve as a basis for revising brake performance standards. The most recent braking performance study, completed in November 1963, is discussed in this article.

Scope of Research

Tests to determine braking performance of motor vehicles operating on the highways were made on foreign, compact, and other (referred to here as standard size) passenger cars; single-unit trucks; and trailer combinations. Vehicles were selected at random from general highway traffic. All vehicles were stopped by a uniformed policeman; they were weighed; the weight and a description were recorded; and three emergency stops were

EDITORIAL INTRODUCTION

In the three articles, *Braking Performance of Motor Vehicles*, the *Relation of Gross Weights and Horsepowers of Commercial Vehicles*, and *Offtracking Calculations for Trailer Combinations*, printed in this issue of the magazine, some common technical terminology is used. For the convenience of the reader, these terms are explained in the following paragraphs. Specific terms are defined in each article.

Single-unit trucks and trailer combinations have been designated by numerical and letter combination codes based on the number of axles and their arrangement. The codes for these commercial vehicles are defined in the following list.

2 =2-axle single-unit truck.
3 =3-axle single-unit truck.
2-S1 =2-axle truck-tractor with 1-axle semitrailer.
2-S2 =2-axle truck-tractor with 2-axle semitrailer.
3-S2 =3-axle truck-tractor with 2-axle semitrailer.
2-1 =2-axle truck with 1-axle trailer.
2-2 =2-axle truck with 2-axle trailer.
3-2 =3-axle truck with 2-axle trailer.
4-6 =4-axle truck-tractor with 6-axle trailer.
2-3 =2-axle truck with 3-axle trailer.
2-S1-2=2-axle truck-tractor with 1-axle semitrailer and 2-axle trailer.
2-S2-2=2-axle truck-tractor with 2-axle semitrailer and 2-axle trailer.
2-S2-3=2-axle truck-tractor with 2-axle semitrailer and 3-axle trailer.
3-S1-2=3-axle truck-tractor with 2-axle semitrailer and 2-axle trailer.
3-S2-2=3-axle truck-tractor with 2-axle semitrailer and 2-axle trailer.
3-S2-4=3-axle truck-tractor with a 2-axle semitrailer and 4-axle full trailer. Also called a double trailer combination.
3-S3-5=3-axle truck-tractor with 3-axle semitrailer and 5-axle trailer.

Other technical terms used in the articles are defined in the following statements.

Gross vehicle weight.—Gross vehicle weight (GVW) is the empty weight, in pounds, including the weight of accessories and fuel, of a passenger car, truck, truck-tractor-semitrailer, or truck-tractor-semitrailer-full trailer combination, plus the weight of the cargo or payload carried at the time the vehicle was tested.

Brake system application and braking distance.—Brake system application and braking distance (BSABD) is the distance, in feet, traveled between the point at which the driver starts to move the braking controls and the point at which the passenger car or commercial vehicle is stopped.

Maximum deceleration.—Maximum deceleration is the peak deceleration measured in percent gravity (1 g.) that occurred during the stopping.

Pedal reserve.—Pedal reserve is the distance, in inches, between the floorboard or mat and the back of the pedal at the completion of a stop.

Brake system air pressure.—The brake system air pressure is air pressure, in pounds per square inch, indicated on the gage in the cab immediately after completion of a stop. This applies to vehicles equipped with some form of an air-actuated brake system. Before any stops were made during this research, the air reservoir was filled by the air compressor.

Manufacturers maximum gross vehicle weight rating.—The manufacturers maximum gross vehicle weight rating is the empty weight, in pounds, of the truck chassis and lubricants, water, fuel tank or tanks of fuel, plus the weight of cab, body, special chassis and body equipment, and the payload recommended by the chassis manufacturer.

Vehicle capacity.—Vehicle capacity for single-unit trucks is the same as the maximum gross vehicle weight rating; for trailer combinations it is the gross combination weight (GCW) recommended by the vehicle chassis manufacturer for a truck-tractor or truck used in combination with semitrailers or full trailers.

Mean.—The mean is a number that represents a set of numbers obtained by dividing the sum of all the numbers or elements in the set by the total number of elements in the set—expressed as: $\bar{X} = \frac{\Sigma X}{N}$

Median.—The median refers to the middle number in a series of test data.

Mode.—The mode is the number in a set of data that occurs most frequently.

Standard deviation.—Standard deviation (S.D.) is the square root of the arithmetic mean of the squares of the deviations from the mean (*1*).[4]

Standard error of the mean.—Standard error of the mean is an estimate of the amount that an obtained mean may be expected to differ by chance from the true mean (*2*).

Gross horsepower.—The gross horsepower of a vehicle is the brake horsepower of the engine available at the clutch or its equivalent, when the engine is being operated but accessories such as fan, air compressor, generator, and muffler are not.

Net horsepower.—Net horsepower is the brake horsepower of the engine available at the clutch or its equivalent, when the engine is being operated with all the normal accessories. In other words, the net horsepower is the gross horsepower minus the horsepower absorbed by accessories such as fan, air compressor, generator, and muffler.

Weight-power ratio.—Weight-power ratio is the ratio of the gross weight of the vehicle or combination of vehicles to net horsepower of the powered unit. For example, if the gross weight of a trailer combination is 60,000 pounds and the net horsepower is 150, the weight-power ratio is 400 pounds per horsepower.

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[1] Presented at the October 1964 meeting of National Transportation, Powerplant, and Fuels and Lubricants Meeting, of the Society of Automotive Engineers, Inc., Baltimore, Md.

[2] Bureau of Public Roads personnel who contributed to the success of the research project include F. William Petring (now with Ford Motor Co.), Harry H. Hill, Harry Krashen, William H. Oliver, John M. Wright, James LoJacono, Richard C. Tennent, William J. Giacofci, Mrs. Madalene Kendall (now retired), John F. Lyons, and Mrs. Ruby Tice.

[3] The Maryland State Roads Commission, the Michigan Department of State Highways, and the California Division of Highways, and personnel from these State organizations assisted in the field research. The Maryland Truck Patrol, the Michigan State Police, the California Highway Patrol, the Sacramento Police Department, and the city of Sacramento also contributed to the success of the project.

[4] References indicated by italic numbers in parentheses are listed on page 82.

made, each from a speed of 20 miles per hour (m.p.h.). Each driver was told that the tests were voluntary and that no punitive action would be taken regardless of the condition of the vehicles brakes. The braking performance was measured in terms of brake system application and braking distance—the distance traveled between the point at which the driver starts to move the braking controls and the point at which the vehicle stops—and in terms of deceleration.

Test sites

The tests were made at four locations: U.S. 40 near Elkton, Md., a 4-lane divided highway; U.S. 24 near Erie, Mich., also a 4-lane divided highway; U.S. 40 near Cordelia, Calif., an 8-lane divided highway; and Elvas Ave., Sacramento, Calif., an undivided city street carrying crosstown traffic. In California, the Cordelia site was used to obtain the commercial vehicle sample, and the Elvas Ave. site was used to obtain the passenger car sample. At each of the other sites, both commercial vehicles and passenger cars were tested. Out-of-State vehicles were tested at each site.

The test section used at each study site was a dry, single-level through lane approximately a half-mile long, separated from other through lanes by rubber traffic cones and/or barricades. Signs were erected that instructed through drivers to merge to a lane other than the test lane and notified them that braking tests were being conducted. Scales were located next to the test lane and were used to determine the gross vehicle weight before the testing. The scales at the sites in Michigan and Cordelia, Calif., were of the permanent pit type used by the States for enforcement of weight regulations. At the other test locations, portable scales were used.

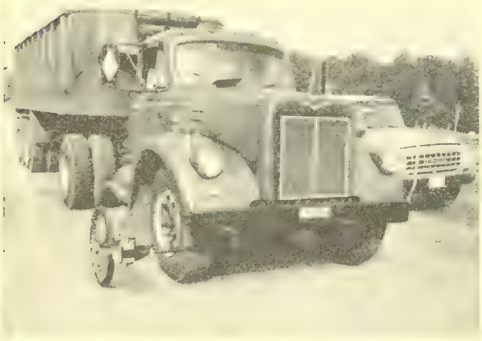

Figure 1.—Trailing fifth-wheel, distance-measuring device attached to a 2-S2.

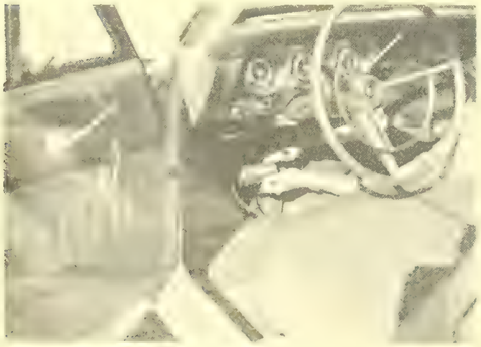

Figure 2.—Brake pedal switch for activating pavement marking and distance measuring devices.

Locked-wheel, passenger-car stops were made at each test section to determine the similarity of the coefficient of friction for the different test surfaces. The results of the locked-wheel stops showed that all of the surfaces had similar frictional characteristics, the average coefficient of sliding friction being 0.82.

Instrumentation

Instrumentation was primarily a test wheel, equipped with a distance measuring device and a portable decelerometer. The test wheel measured the speed of the test vehicle in miles per hour and the brake system application and braking distance in feet. The decelerometer measured the maximum deceleration occurring during the braking test in percent of 1 g. The instrumentation is shown in figures 1 through 4.

The test wheel (fig. 1), referred to as a fifth wheel, was equipped to start the distance-measuring device when the driver touched his foot to a switch attached to the brake pedal (fig. 2). When the driver's foot first touched the brake pedal switch, an electrical circuit was completed and it was maintained by a holding relay until released by the observer. An observer, who rode with the test driver, recorded the distance shown on the dial of the distance recording device, the speed, the deceleration, and the other information relevant to the stop.

Speed of the test vehicle was measured in miles per hour by a voltmeter wired to a belt-driven generator, which was mounted on the frame of the test wheel (fig. 3). The observer held the voltmeter (fig. 4) and when a speed of 20 m.p.h. was reached, he told the driver to stop. A pendulum-type decelerometer (fig. 4) was used. A moving scale, indicating percent of 1 g., was actuated by, and proportional to, the movement of a pendulum. When the test vehicle moved at a uniform speed, the pendulum assumed a vertical position; but when the speed was reduced by the application of brakes, the pendulum tended to move at the initial speed and thus swing forward. The tangent of the angle through which the pendulum moved from its vertical position was proportional to the deceleration. A scale reading of 80 percent thus would reflect a deceleration of 0.80×32.2, or 25.8 feet per second per second (ft./sec./sec.). The decelerations measured by a pendulum-type decelerometer are often larger than actual decelerations, which can be measured by more sophisticated equipment. The pendulum-type decelerometer, however, is effective for identifying vehicles that have improperly maintained brakes.

All equipment used for the braking performance tests was calibrated frequently during the tests to assure accuracy of test results. The speedometer or voltmeter was calibrated by measuring with the test wheel the time required to travel a measured mile at a constant speed. The accuracy of the distance-measuring device was verified by use of an electric detonator mounted on the bumper. The detonator ejected a chalk capsule that marked the pavement at the instant the driver touched his foot to the brake switch pedal—the same switch that activated the distance measuring device. The brake system application and braking distance shown on the dial for the test wheel was compared with the distance measured between the chalk mark on the pavement and a point below the detonator on the test vehicle. To calibrate the moving scale on the decelerometer, the instrument was placed on a known slope and the tangent of the slope compared with the scale reading. These periodic tests of equipment showed a variation of 2 percent or less between the test and theoretical results.

Test procedures

When a vehicle was selected for testing, the driver was directed, by a uniformed officer, from the through lanes into an interview and pit area adjacent to the test lane. The test procedures were explained to each driver and those preferring not to participate in the tests were permitted to continue.

On one of two cards, both having the same test number, the vehicle characteristics were recorded. The information noted included vehicle type, make, model, year, type of transmission, tire size, type of cargo, manufacturers maximum gross vehicle weight rating, type of brake system, and number of

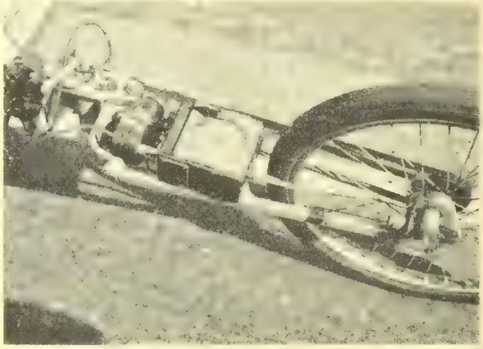

Figure 3.—Dial on fifth wheel for measuring brake system application and braking distance.

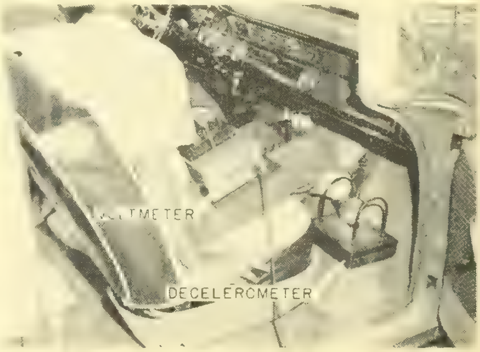

Figure 4.—Placement of test instrumentation.

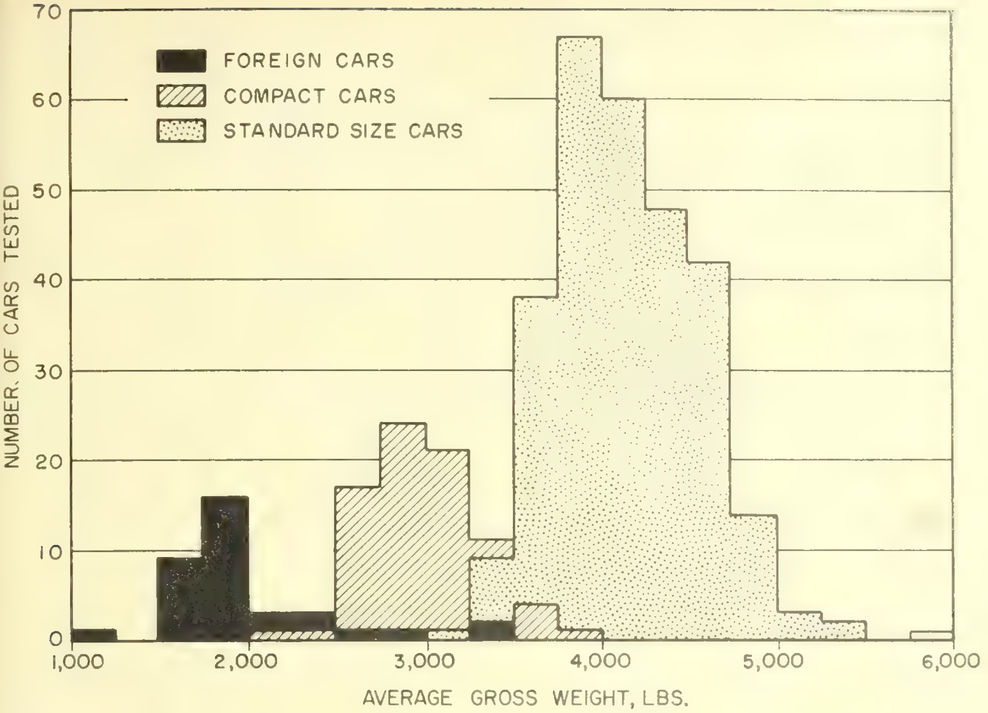

Figure 5.—Passenger car weight distributions.

unbraked axles; this card also had space for recording test data. On the other card, the vehicle weight by individual axle was recorded. The equipment for measuring the brake performance then was installed.

An observer seated next to the driver directed him to the test lane. Before any tests were conducted, the driver was told to disengage the clutch during the stop and, if it were a commercial vehicle, to set the limiting valve in the dry road position and not to use the hand control valve during the tests. Approximately three emergency-type stops were made, each from a speed of 20 m.p.h. Each stop was made upon the observer's direction, when the test speed had been reached. The driver applied the brakes and maintained the vehicles maximum braking capacity. After each stop, the observer recorded the brake system application and braking distance, the maximum deceleration, and the pedal reserve or brake system air pressure.

As the test lane was separated from all through traffic by rubber cones, barricades, or both, an unobstructed lane was available for each test vehicle. Thus the 20 m.p.h. speed could be stabilized for the braking stops. To prevent interference from a vehicle inadvertently entering the test lane, a project vehicle—equipped with a flashing red light on top and a large sign mounted on the rear that stated DANGER–BRAKE TESTS–SUDDEN STOP— followed the test vehicle. An observer also was in the project vehicle; he measured the lengths of any skid marks that were left by the test vehicle and entered these lengths on the weight data card. When the braking tests had been completed, the test equipment was removed from the vehicle and the driver was thanked for his cooperation. The equipment then was returned to the pit area for use on the next test vehicle.

Analyses

In the analyses of test data, vehicles were classified according to vehicle type; capacity, based on manufacturers maximum vehicle weight ratings; test gross weights; and brake type. The braking performance results from the 1963 study were compared with the performance requirements of the *Uniform Vehicle Code* (*5*) and with the results of the previous studies. Statistical tests were performed at different points in the analyses to determine whether the difference in observed means was statistically significant and to obtain insight into the meaning of the results. Separate analyses were made for the passenger car and commercial vehicle test results; these analyses are presented separately. For both analyses, a 40K7010 IBM computer was used in different phases. The computer arranged the vehicles by State, vehicle type, brake type, manufacturers capacity weight rating groups, and test gross weight. In general, its use expedited the computation of statistics such as means, standard deviations, and confidence intervals.

Passenger Cars

Passenger cars tested were classified as foreign, compact, and standard size. Because of the increase in popularity of the foreign and compact passenger cars and the frequency of their operation on the highways, an analysis of the braking performance of such vehicles was considered desirable. Also data for the foreign, compact, and standard size cars were combined and analyzed for comparison with the results of previous studies.

Any passenger car produced in a country outside the United States was placed in the foreign car category. Passenger cars not included in either the foreign or compact categories were classified as standard size cars. The criteria used to classify compact cars included primarily, the make of vehicle, year of production, gross weight, wheelbase, overall length, and horsepower, according to the method described by Cope and Liston (*3*). Automobile insurance company guides also were consulted.

The weight distributions for the foreign, compact, and standard size cars are shown in figure 5. Although some overlap of classi-

Table 1.—Passenger cars tested and results of analyses of data by classification from 1963 braking study

Data analyses	Foreign	Compact	Standard	Total
Gross weight:				
Number of cars	37	80	285	402
Mean..........pounds	2,040	2,996	4,164	3,736
S.D..........do	477	319	409	815
Mode..........do	1,875	2,875	3,875	3,875
Deceleration:				
Number of cars [1]	37	80	283	400
Mean..........feet/second/second	28.6	29.7	29.3	29.3
S.D..........do	2.73	2.45	2.76	2.70
Mode..........do	30.6	32.2	32.2	32.2
Distance:				
Number of cars	37	80	285	402
Mean..........feet	19.3	19.0	20.0	19.7
S.D..........do	2.65	1.16	2.09	2.04
Mode..........do	19	20	19	19

[1] Number of cars shown does not always agree with number in weight and brake system application and braking distance columns as some of the cars did not have enough room for installation of the decelerometer.

Table 2.—Confidence interval for passenger cars classifications in which population means would be expected 95 percent of the time

Data analyses	Foreign	Compact	Standard	Total
Gross weight:				
Standard error of mean..lbs	78.45	35.68	24.23	40.65
95 pct. confidence interval..do	1,890–2,190	2,930–3,070	4,120–4,210	3,660–3,820
Deceleration:				
Standard error of mean, feet/second/second	0.449	0.274	0.164	0.135
95 pct. confidence interval..do	27.7–29.5	29.2–30.2	29.0–29.6	29.0–29.6
Distance:				
Standard error of mean..feet	0.436	0.130	0.124	0.102
95 pct. confidence interval..do	18.4–20.2	18.7–19.3	19.8–20.2	19.5–19.9

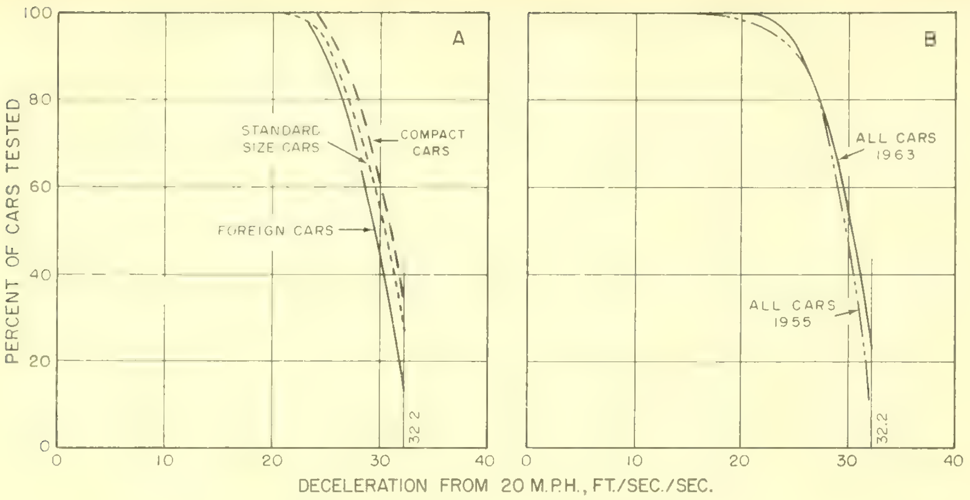

Figure 6.—Passenger cars decelerations.

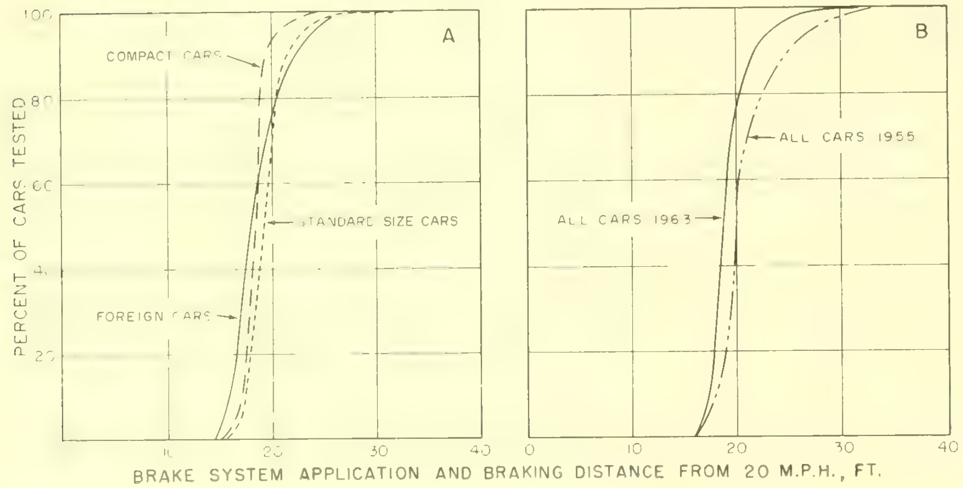

Figure 7.—Frequency distribution of brake system application and braking distances for passenger cars.

fications existed, it constituted less than 5 percent of the 402 passenger cars tested. To determine whether the passenger car classifications were significantly different, an analysis of variance was performed. The null hypothesis was formulated that no difference existed between the average test weight for the foreign, compact, and standard size passenger car classifications. A level of significance of 0.05 was used; in other words, about 5 chances in 100 existed that the hypothesis would be rejected when it should be accepted. The mean test weights were determined to have been unequal and sufficiently different to require individual analysis for each classification: each passenger car classification, the number of vehicles tested, the average or mean value, the standard deviation, and the mode for the gross vehicle test weight, deceleration, and brake system application and braking distance are listed in table 1.

Braking Performance

Use of cumulative frequency distribution curves is a convenient method for comparing the relative performance of the different classifications of passenger cars. Frequency distributions for deceleration and for brake system application and braking distances for each passenger car classification are shown in figures 6 and 7.

The frequency distribution for passenger car deceleration is shown in figure 6. In part B little change is shown to have occurred in the deceleration performance of all passenger cars since 1955 (*4*), however, in the 1963 tests nearly 16 percent more cars than in the 1955 study reached a peak deceleration of 1 g. The average deceleration for each passenger car classification was compared at the 0.05 level to determine whether the differences in means were statistically significant Only the foreign car comparison with the compact car showed significantly different decelerations.

The frequency distribution curve in figure 7 indicates the percentage of passenger cars capable of stopping in a given brake system application and braking distance from a speed of 20 m.p.h. In general the 1963 test results were better than those obtained in 1955, particularly above the 50th percentile level, as shown in part B. In part A of figure 7, data show a larger variability in the distances for the foreign cars than for the compact cars. This difference in variability is also shown in table 1; the standard deviation of the brake system application and braking distance for the foreign car classification exceeds that for the compact car.

An analysis of the average of test data of the brake system application and braking distance for each passenger car classification was made and compared with the average for each of the other classifications. The means for the compact cars and standard size cars differed significantly at the 0.05 level. The analysis of the data for foreign cars compared with that for compact cars and of the data for foreign cars compared with that for standard size cars showed no significant differences at the 0.05 level; thus no real difference existed between the means of brake system application and braking distance tests for these two passenger car classifications.

The braking performance of passenger cars had improved since the first series of tests were made in 1942 (*5*). The test data show a general reduction in the variability of the brake system application and braking distance between 1942 and 1963; the results from the 1963 tests had one-fifth the variability of the 1942 results, although the 1942 results included data for some passenger cars equipped with mechanical-type braking systems. The general reduction in braking performance results at the 85th, 50th, and 15th percentiles are shown in figure 8 for the studies made in 1942, 1949, 1955, and 1963.

Of the passenger cars tested, only a few in the compact and foreign classifications had vacuum power brakes, but 93 of the 285 standard size passenger cars tested had vacuum power brakes. The average brake system application and braking distance for cars that had vacuum power brakes was 20.1 feet compared with 19.9 feet for cars that had regular hydraulic systems. Comparison at the 0.05 level, showed no real or significant statistical differences in the mean braking performance of the two systems.

Uniform Vehicle Code

The National Committee on Uniform Traffic Laws and Ordinances presently recommends in its *Uniform Vehicle Code* (*5*) that all passenger cars stop in 25 feet or less from a speed of 20 m.p.h. As computed, nearly 97 percent of the passenger cars tested in 1963 stopped in 25 feet or less. At the 95th percentile level the passenger cars stopped in 23.5 feet and at the 85th percentile level passenger cars stopped in approximately 22 feet. The committee also recommends that all passenger cars decelerate from a speed of 20 m.p.h. in not less than 17 ft./sec./sec. As indicated by the pendulum-type decelerometer the smallest peak deceleration was 17.7 ft./sec./sec. and computed in the analysis, 95 percent of the passenger cars could stop with a peak deceleration of more than 24.1 ft./sec./sec. From the results of the 1963 brake performance test, the *Uniform Vehicle Code* (*5*) seems to be liberal. Perhaps the code requirements should be updated to encourage additional improvement in the overall braking performance of passenger cars.

Age of vehicle

An analysis was conducted on 285 standard size passenger cars equipped with either vacuum power brakes or regular hydraulic brakes to determine whether the average brake system application and braking distance varied with the age of the car. To determine whether the brake system application and braking distances means were significantly different, an analysis of variance was performed. The null hypothesis was formulated that no difference existed between the mean braking distance regardless of the age of the vehicle. A level of significance of 0.01 was used, and the mean brake system application and braking distances for the different years were significantly different at that level.

A linear regression equation that best fit the data was computed by the method of least squares. This linear regression is shown in figure 9. The coefficients of correlation and the coefficients of determination were also computed. The coefficient of correlation (r) is a measure of the goodness of fit of the regression equation to the data; 1.00 indicates a perfect fit and 0.00 indicates no fit (*6*). The coefficient of determination (r^2), the square of the coefficient of correlation, represents the part of the total variance that can be accounted for by the independent variable, which here is the age of vehicle (*6*).

The coefficient of correlation of 0.28 indicated that the regression curve did not fit the data as well as it might have. The coefficient of determination indicated that only 8 percent of the total variation in the brake system application and braking distance can be attributed to the age of the passenger car. The remaining or unexplained variation must be attributed to other factors such as inadequate brake system maintenance and/or poor brake adjustment.

The fact that a large percentage of the variability in brake system application and braking distance is unexplained, also is illustrated in figure 9. The 95 percent confidence interval is shown in figure 9 by parallel lines 3.94 feet above and below the regression line $Y=0.149x+19.44$. For example, if the brake system application and braking distance is to be estimated on the basis of age, for standard size passenger cars 5 years old; the distance would be expected to fall within the interval of 16.25 to 24.13 feet, 95 percent of the time. This large interval emphasizes that the age of a passenger car is not by itself a good parameter for estimating braking performance.

Confidence intervals

The classification of passenger cars, as previously explained, represents samples of the passenger cars operating on the public highways. In evaluating the braking performance of the entire population of cars within each classification, the means for each sample classification were used to determine the interval in which the population mean could be expected to fall with some degree of confidence. The confidence interval selected was 95 percent; meaning if 100 samples were taken from the population, 95 of the sample means would be within the computed interval. In computing the confidence interval, the standard error of the mean was adjusted for a probability of 0.95. The equation used for determining the confidence interval for the population mean was:

95 percent confidence interval=sample mean ± 1.96 × standard error of the mean

The interval in which the population mean for gross weight, deceleration and the brake system application and braking distance could be expected 95 percent of the time for the different classifications of passenger cars is shown in table 2.

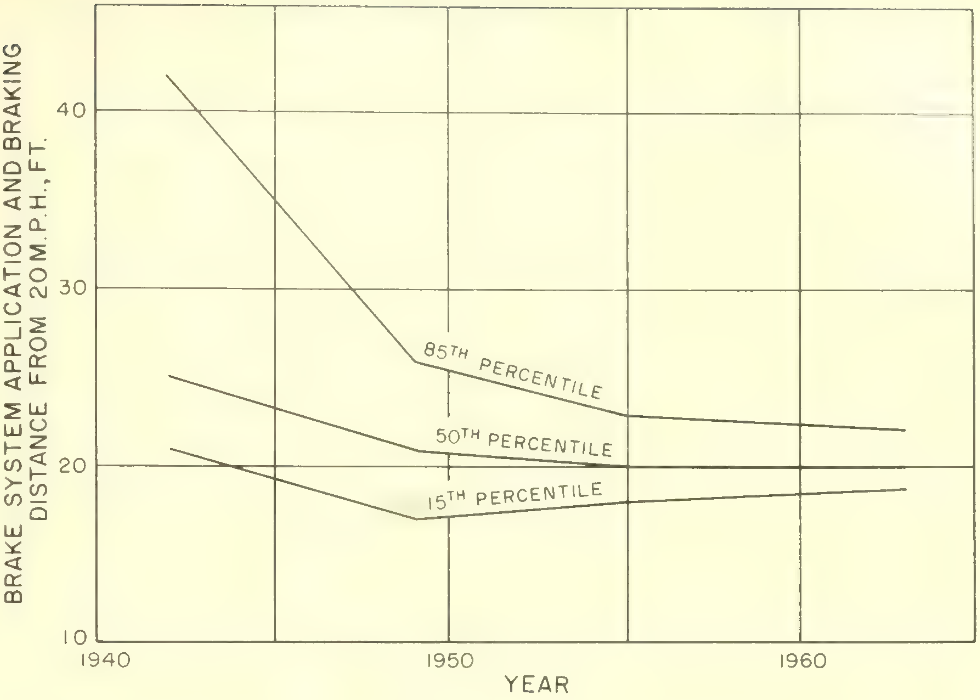

Figure 8.—Percentile levels of brake system application and braking distances for passenger cars.

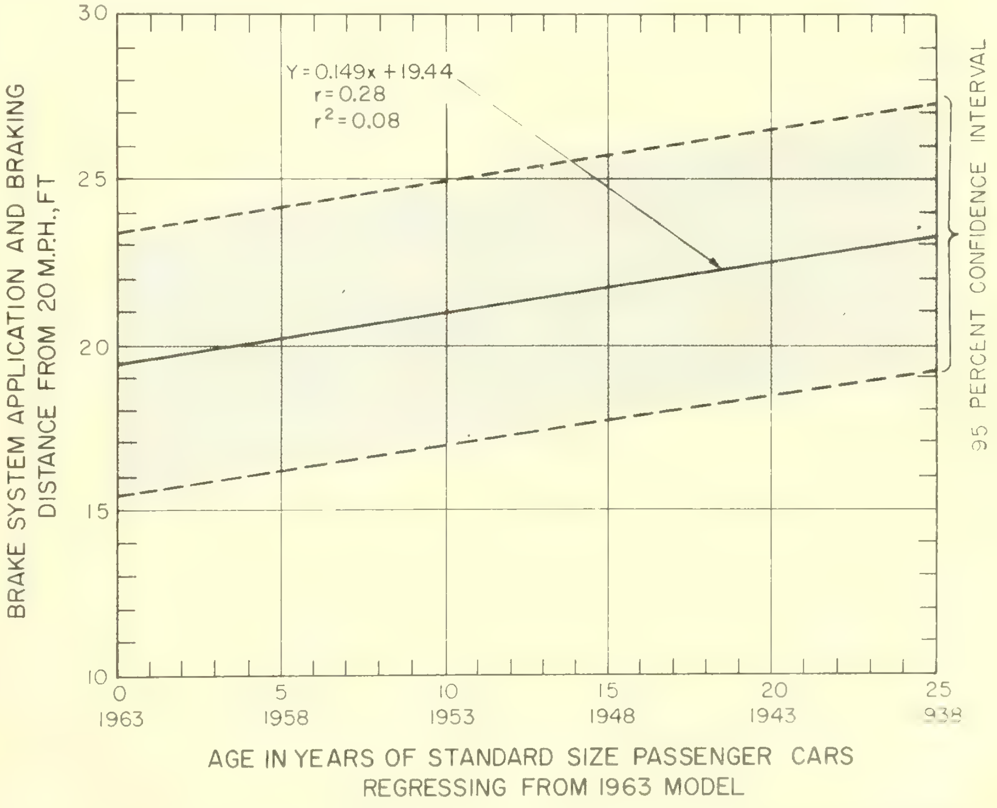

Figure 9.—Mean brake system application and braking distances by age of passenger car.

Commercial Vehicle Test Results

The commercial vehicles tested were grouped according to vehicle type, capacity group, and brake type. Results from tests made with similar or like vehicles could then be considered together and the braking performance determined for the respective groupings. Types of commercial vehicles are shown in figure 10.

Capacity Groups

All commercial vehicles were classified by capacity groups on the basis of the chassis manufacturers gross vehicle weight or gross combination weight rating as marked on the rating plate attached to the test vehicle. Single-unit trucks were classified as very light, light, medium, and heavy; trailer combinations were classified as light, medium, and heavy. The distribution of gross weight ratings by capacity groups is shown in table 3. Sometimes the chassis manufacturers maximum gross weight for truck or truck-tractor used in combination with trailers was not available on the vehicles tested. These trailer combinations were classified as light, medium, or heavy on the basis of the power unit when it is used as a single-unit truck.

Brake Types

Four types of braking systems are commonly used on single-unit vehicles: hydraulic, vacuum-booster hydraulic, air-booster hydraulic, or air-mechanical systems. On trailer combinations the power units are braked by vacuum-booster hydraulic, air-booster hydraulic, or air-mechanical systems. The semitrailers and full trailers within the trailer combination generally are braked by air-mechanical or vacuum-mechanical systems. The brake types used on the vehicles tested are defined as:

Hydraulic (*H*).—Hydraulic brakes have brake shoes that are actuated by hydraulic-brake cylinders operated with hydraulic-line pressure developed by a pedal-operated hydraulic brake master cylinder.

Vacuum-booster hydraulic (*VBH*).—Vacuum-booster hydraulic brakes have brake shoes that are actuated by hydraulic brake wheel cylinders operated with hydraulic-line pressure developed by a vacuum-powered master cylinder or a vacuum-hydraulic power unit.

Air-booster hydraulic (*ABH*).—Air-booster hydraulic brakes have brake shoes that are actuated by hydraulic brake wheel cylinders operated with hydraulic-line pressure developed by an air-powered master cylinder or an air-hydraulic power unit.

Air mechanical (*AM*).—Air mechanical brakes have brake shoes that are actuated by a cam or wedge operated by an air-brake chamber through a mechanical linkage.

Vacuum-mechanical (*VM*).—Vacuum-mechanical brakes have brake shoes that are actuated by a cam or wedge operated by a vacuum-brake chamber through a mechanical linkage.

A code also was used to represent the system or systems employed in braking the vehicles. Each individual part of the code represents the braking system used in a single-unit truck or in one unit of a trailer combination. A combination code consisting of two or three parts separated by hyphens indicates the braking system used in each unit of the trailer combination. For example, a truck-tractor, semitrailer, and full trailer combination having a braking code of VBH–VM–VM would have vacuum-booster hydraulic brakes on the truck-tractor and vacuum-mechanical brakes on both the semitrailer and full trailer.

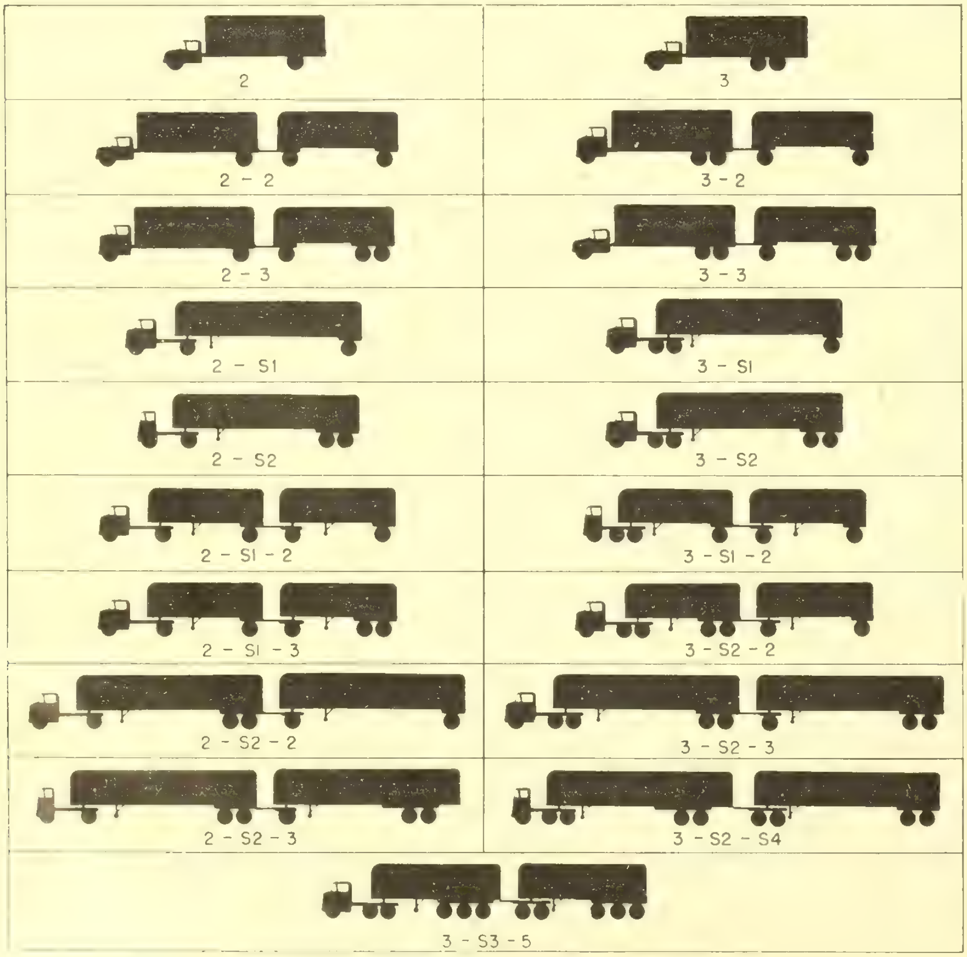

Figure 10.—Commercial vehicles.

Vehicle Sample Size

Approximately 300 commercial vehicles were tested in each of the three States. In each State the sample was composed of nearly 50 percent single-unit vehicles and 50 percent combination vehicles. Test vehicles were chosen at each test site so as to obtain a sample in which gross vehicle or gross combination weights were distributed as evenly as possible from the lightest to heaviest weights. In table 4, the number of vehicles tested in each State are shown by type, capacity group, and brake type. No truck-tractors with semitrailers and full trailers were tested in Maryland because none came along during the testing period.

Weight, Deceleration, and Distance Observations

Tables 5 and 6 show for each commercial vehicle grouping the number of vehicles tested, the mean, the standard deviation, and the minimum and maximum test results for gross vehicle weight, deceleration, and the brake system application and braking distance. Both tables present the results by vehicle type and capacity group: table 5 shows the results by type of brake system and table 6 by weight groups. For example, the mean brake system application and braking distance for the heavy capacity, 2-axle, single-unit trucks braked by air-mechanical systems (AM) was 31.7 feet; and the mean distance—without regard to the type of brake system—for the heavy capacity-2-axle, single-unit trucks having a gross vehicle weight between 10,000 and 20,000 pounds was 29.7 feet.

The minimum and maximum results for for gross weight, deceleration, and braking distance in the two tables should not be specifically associated with each other. These results only indicate the spread of the data for each individual parameter; they are the extremes and define the low and high limits. Minimum and maximum results for one parameter, such as deceleration, cannot be associated with the corresponding results for

Table 3.—Capacity group classifications of commercial vehicles by manufacturers ratings

Capacity group	Manufacturers gross weight rating	
	Single-unit trucks	Trailer combinations
	Pounds	*Pounds*
Very light	10,000 and less	
Light	10,001–16,000	27,000 and less.
Medium	16,001–24,000	27,001–44,000.
Heavy	24,000 and more	44,000 and more.

Table 4.—Classification of vehicles tested by type, capacity group, and brake type

Commercial vehicles and capacity group	Brake type	Number of vehicles tested in			
		Md.	Mich.	Calif.	Total
Single-unit trucks.					
2-axle:					
Very light	H	46	37	33	116
	VBH	1	2		3
Light	H	4	7	3	14
	VBH	17	26	21	64
Medium	H	2	4		6
	VBH	55	65	51	171
	ABH		2	2	4
	AM	1	7	2	10
Heavy	VBH	7	4		11
	AM	12	2	2	16
3-axle:					
Light	VBH			1	1
Medium	VBH		2	2	4
	AM		1	1	2
Heavy	VBH	1	2	2	5
	AM	11	6	14	31
Truck-tractors with semitrailers:					
2-S1.					
Medium	VBH-VM	6	11	1	18
	ABH-AM		1	1	2
	AM-AM	5	8	8	21
	ABH-VM	...	1		1
	AM-VM		1		1
Heavy	VBH-VM	2	1		3
	ABH-AM	3			3
	AM-AM	15	15	22	52
	AM-VM	1	...	1	2
2-S2:					
Medium	VBH-VM	...	1		1
	ABH-AM		1	3	4
	AM-AM	7	6	....	13
Heavy	ABH-AM		1	1	2
	AM-AM	104	64	10	178
2-S3:					
Medium	AM-AM	...	1		1
Heavy	AM-AM		1	...	1
3-S2:					
Medium	AM-AM	...	1	...	1
Heavy	ABH-AM	...	1		1
	AM-AM	22	45	31	98
Trucks with full trailers:					
2-2:					
Heavy	AM-AM	...	2	..	2
3-2:					
Heavy	AM-AM		1	25	26
Truck-tractors with semitrailers and full trailers:					
2-S1-2:					
Medium	VBH-VM-VM.	...	...	1	1
Heavy	AM-AM-AM.		8	40	48
2-S2-2:					
Medium	AM-AM-AM.		1		1
Heavy	AM-AM-AM.		4	..	4
2-S2-3:					
Heavy	AM-AM-AM.	....	4		4
3-S1-2:					
Heavy	AM-AM-AM.	...	...	1	1
3-S2-2:					
Heavy	AM-AM-AM.	...	2		2
3-S3-5:					
Heavy	AM-AM-AM.	...	2		2
Other combinations:					
Truck driveaway-towaway:					
Heavy	VBH		1	..	1
	AM	...	1		1
Housetrailer factory towaway:					
Medium	VBH-E	2	1	1	4
Single-unit trucks with unbraked utility trailer:					
Medium	VBH	...	3	..	3
TOTAL		324	357	280	961

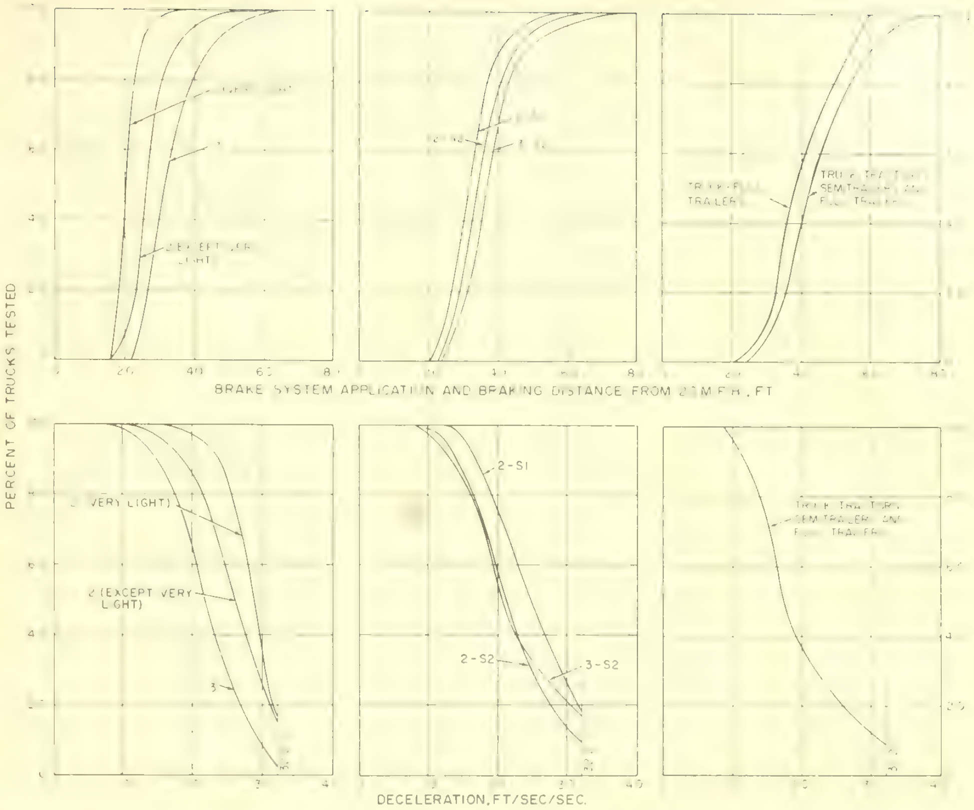

Figure 11.—Cumulative frequency distributions of minimum brake system application and braking distances and decelerations.

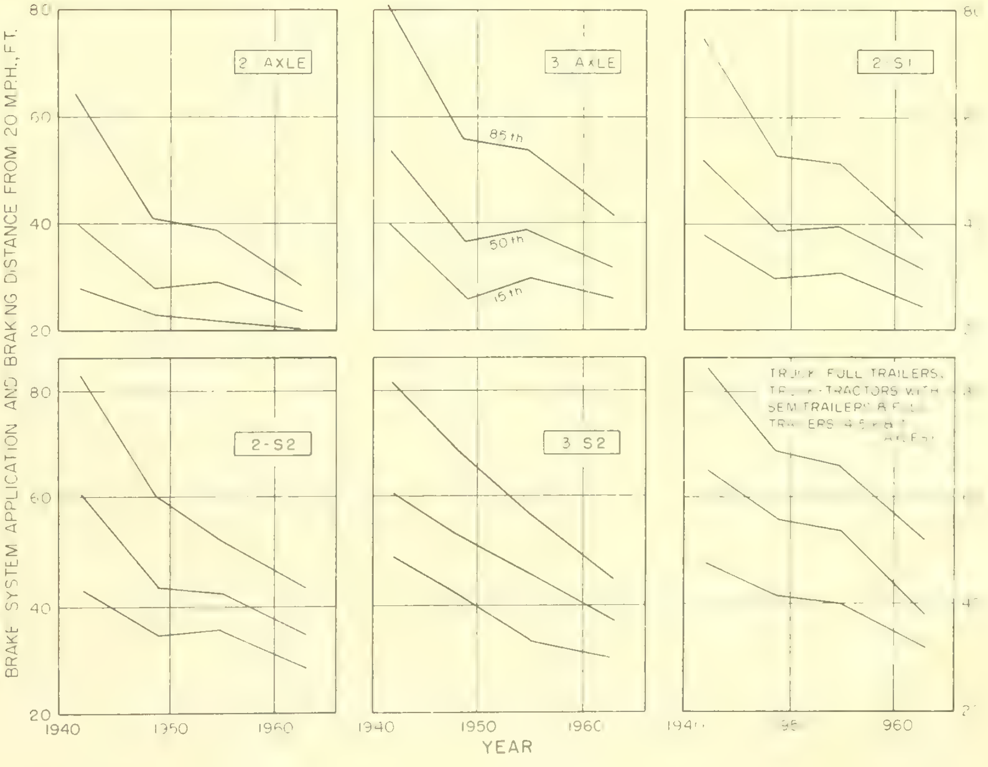

Figure 12.—Percentile levels of brake system application and braking distances for vehicles by year.

Table 5.—Analysis results for gross vehicle weight, braking system application and braking distance, and deceleration, by type of brake system

Commercial vehicles and capacity groups		Brake system[1]	Test vehicles, GVW and BSABD	Gross vehicle weight				BSABD				Deceleration (pendulum-type decelerometer)				
				Mean	Standard deviation	Minimum	Maximum	Mean	Standard deviation	Minimum	Maximum	Test vehicles	Mean	Standard deviation	Minimum	Maximum
		Type	*No.*	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Feet*	*Feet*	*Feet*	*Feet*	*No.*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*
Single-unit trucks:																
2-axle	Very light	H	116	4,687	1,110	2,545	8,700	21.6	3.65	16	41	116	27.6	3.81	15.1	32.2
		VBH	3	7,918		6,320	11,050	21.0	---	19	23	3	29.8	---	28.3	32.2
		All	119	4,768	1,259	2,545	11,050	21.6	3.62	16	41	119	27.7	3.78	15.1	32.2
	Light	H	14	10,126	4,164	5,700	18,530	29.4	7.31	18	40	14	21.9	5.17	14.5	30.6
		VBH	64	10,668	4,466	5,900	23,840	26.4	5.82	19	51	63	26.5	5.68	11.9	32.2
		All	78	10,571	4,392	5,700	23,840	27.0	6.17	18	51	77	25.7	5.84	11.9	32.2
	Medium	H	6	12,239		10,205	15,660	28.5	---	26	31	5	25.2	---	20.9	32.2
		VBH	171	13,635	4,809	6,710	27,400	27.4	5.49	17	46	167	25.5	4.87	12.2	32.2
		ABH	4	12,155	---	8,900	18,570	34.7	---	26	46	4	24.3	---	15.1	27.4
		AM	10	17,544	6,382	8,100	26,800	37.8	16.82	24	75	10	18.8	7.53	6.4	29.0
		All	191	13,765	4,892	6,710	27,400	28.1	6.95	17	75	186	25.1	5.23	6.4	32.2
	Heavy	VBH	11	15,777	5,292	10,605	28,480	30.4	5.79	24	44	11	27.6	4.72	18.0	32.2
		AM	16	17,936	5,249	12,410	31,410	31.7	5.40	25	46	15	22.0	3.54	14.5	30.6
		All	27	17,056	5,276	10,605	31,410	31.2	5.49	24	46	26	24.3	4.91	14.5	32.2
	All capacity groups	H	136	5,580	2,768	2,545	18,530	22.7	4.87	16	41	135	26.9	4.33	14.5	32.2
		VBH	249	12,898	4,943	5,900	28,480	27.2	5.64	17	51	244	25.9	5.09	11.9	32.2
		AM	26	17,785	5,589	8,100	31,410	34.2	11.33	24	75	25	20.7	5.57	6.4	30.6
		All	415	10,799	5,818	2,545	31,410	26.2	6.66	16	75	408	25.9	5.08	6.4	32.2
3-axle	Light	VBH	1	25,400	---	25,400	25,400	28.0	---	28	28	1	23.2	---	23.2	23.2
	Medium	VBH	4	15,808	---	11,000	23,400	28.7	---	27	30	4	24.5	---	22.5	27.4
		AM	2	26,725	---	17,500	35,950	45.0	---	28	62	2	18.5	---	11.3	25.8
		All	6	19,447	---	11,000	35,950	34.1	---	27	62	6	22.5	---	11.3	27.4
	Heavy	VBH	5	15,491	---	12,925	17,900	25.4	---	23	29	5	27.4	---	22.5	32.2
		AM	31	25,566	11,120	12,300	53,200	35.9	9.56	24	68	31	19.8	4.61	8.0	30.6
		All	36	24,166	10,900	12,300	53,200	34.5	9.63	23	68	36	20.8	5.26	8.0	32.2
	All capacity groups	VBH	10	16,609	4,702	11,000	23,400	27.0	2.45	23	30	10	25.8	3.74	22.5	32.2
		AM	33	25,636	11,014	12,300	53,200	36.5	10.42	24	68	33	19.7	4.83	8.0	30.6
		All	43	23,537	10,586	11,000	53,200	34.3	10.03	23	68	43	21.1	5.25	8.0	32.2
Truck-tractors with semi-trailers:																
2-S1	Medium	VBH-VM	18	22,602	7,503	13,000	37,000	34.0	8.20	24	56	18	24.3	5.69	12.2	32.2
		ABH-AM	2	16,850		16,800	16,900	26.5	---	25	28	2	27.4	---	25.8	29.0
		AM-AM	21	22,822	6,310	15,300	33,980	31.1	6.07	24	53	19	26.8	5.89	17.4	32.2
		ABH-VM	1	21,610	---	21,610	21,610	29.0	---	29	29	1	22.5	---	22.5	22.5
		AM-VM	1	32,840		32,840	32,840	37.0	---	37	37	1	22.5	---	22.5	22.5
		All	43	22,657	6,772	13,000	33,980	32.2	7.01	24	56	41	25.4	5.64	12.2	32.2
	Heavy	VBH-VM	3	38,848		34,680	41,455	40.0	---	35	46	3	17.4	---	14.5	19.3
		ABH-AM	3	19,800		19,490	20,170	27.0		24	29	2	27.7	---	25.8	29.6
		AM-AM	52	25,748	7,524	14,500	45,400	33.2	6.76	21	51	52	23.6	6.13	12.2	32.2
		AM-VM	2	18,753		17,400	20,105	32.5		32	33	2	27.4		22.5	32.2
		All	60	25,872	7,850	14,500	45,400	33.2	6.72	21	51	59	23.5	6.11	12.2	32.2
	All capacity groups	VBH-VM	21	24,923	9,117	13,000	41,455	34.9	8.05	24	56	21	23.3	5.85	12.2	32.2
		ABH-AM	5	18,620		16,800	20,170	26.8	---	24	29	4	27.5	---	25.8	29.6
		AM-AM	73	24,906	7,276	14,500	45,400	32.6	6.59	21	53	71	24.4	6.19	12.2	32.2
		AM-VM	3	23,448		17,400	32,840	34.0		32	37	3	25.2	---	20.9	32.2
		All	103	24,530	7,554	13,000	45,400	32.8	6.82	21	56	100	24.3	5.97	12.2	32.2
2-S2	Medium	VBH-VM	1	42,080	---	42,080	42,080	48.0	---	48	48	1	12.9		12.9	12.9
		ABH-AM	3	29,783		15,900	49,950	33.6	---	26	40	3	22.0		17.7	27.4
		AM-AM	14	38,409	16,588	17,840	63,390	34.2	7.18	25	45	14	22.2	6.16	14.5	30.6
		All	18	37,175	16,134	15,900	63,390	34.8	7.49	25	48	18	21.7	6.06	12.9	30.6
	Heavy	ABH-AM	2	22,445		21,200	23,690	25.0	---	24	26	2	29.8		27.4	32.2
		AM-AM	178	39,473	13,008	18,500	64,805	36.1	7.66	23	67	177	21.4	6.04	8.7	32.2
		All	180	39,283	13,059	18,500	64,805	36.0	7.70	23	67	179	21.5	6.08	8.7	32.2
	All capacity groups	ABH-AM	5	26,848		15,900	49,900	30.2	---	24	40	5	25.1		17.7	32.2
		AM-AM	192	39,395	13,252	17,840	64,805	36.0	7.62	23	67	191	21.5	6.04	8.7	32.2
		All	198	39,092	13,334	15,900	64,805	35.9	7.67	23	67	197	21.5	6.06	8.7	32.2
2-S3	Medium	AM-AM	1	22,680		22,680	22,680	38.0	---	38	38	1	20.0	---	20.0	20.0
	Heavy	AM-AM	1	42,480	---	42,480	42,480	45.0	---	45	45	1	17.1	---	17.1	17.1
	All capacity groups	AM-AM	2	32,580	---	32,580	32,580	41.5	---	41	41	2	18.5	---	18.5	18.5
3-S2	Medium	AM-AM	1	72,480	---	72,480	72,480	68.0		68	68	1	9.7	---	9.7	9.7
	Heavy	ABH-AM	1	24,090	---	24,090	24,090	38.0		38	38	1	32.2		32.2	32.2
		AM-AM	98	50,345	18,769	22,010	94,650	37.9	9.27	24	79	97	21.7	6.94	8.7	32.2
		All	99	50,079	18,858	22,010	94,650	37.9	9.22	24	79	98	21.8	6.98	8.7	32.2
	All capacity groups	AM-AM	99	50,568	18,805	22,010	94,650	38.2	9.70	24	79	98	21.6	7.01	8.7	32.2
		All	100	50,303	18,896	22,010	94,650	38.2	9.65	24	79	99	21.7	7.06	8.7	32.2
Trucks with full trailers:																
2-2	Heavy	AM-AM	2	42,795	---	24,310	61,280	41.5	---	35	48	2	18.2	---	12.9	23.5
3-2	Heavy	AM-AM	26	48,969	23,793	22,400	78,200	41.3	9.48	23	60	26	23.1	8.95	11.3	32.2
Truck-tractors with semi-trailers and full trailers:																
2-S1-2	Medium	VBH-VM-VM	1	25,600		25,600	25,600	49.0		49	49	1	17.7	---	17.7	17.7
	Heavy	AM-AM-AM	48	60,551	21,871	24,500	82,700	46.5	11.71	27	75	48	19.1	6.79	9.3	32.2
	All capacity groups	All	49	59,837	22,211	24,500	82,700	46.5	11.59	27	75	49	19.1	6.73	9.3	32.2
2-S2-2	Medium	AM-AM-AM	1	77,430		77,430	77,430	53.0		53	53	1	12.9	---	12.9	12.9
	Heavy	AM-AM-AM	4	74,853		35,010	96,260	49.8		30	64	4	18.7	---	10.6	29.0
	All capacity groups	AM-AM-AM	5	75,368		35,010	96,260	50.4		30	64	5	17.5		10.6	29.0
2-S2-3, Heavy		AM-AM-AM	4	88,768		38,440	113,460	40.5		37	49	4	21.3		17.7	25.8
[illegible]-S1-2, Heavy		AM-AM-AM	1	52,200	---	52,200	52,200	31.0		31	31	1	17.4		17.4	17.4
[illegible]-2, Heavy		AM-AM-AM	2	37,010		34,970	39,030	37.0		37	37	2	29.9	---	27.4	32.2
3-[illegible]-5, Heavy		AM-AM-AM	2	132,535		132,500	132,570	58.5		48	69	2	13.8		11.6	16.1
All other than 2-S1-2:																
	Medium	AM-AM-AM	1	77,430		77,430	77,430	53.0	---	53	53	1	12.9		12.9	12.9
	Heavy	AM-AM-AM	13	80,442	36,952	34,970	132,570	44.8	12.27	30	69	13	20.3	6.55	10.6	32.2
	All capacity groups	AM-AM-AM	14	80,227	35,511	34,970	132,570	45.4	11.99	30	69	14	19.8	6.60	10.6	32.2

---

[1] All refers to the total number of vehicles tested in each category, regardless of brake system type or capacity group.

the other two. For example, in the classification under truck-tractors with semitrailers and full trailers and for all other than 2–S1–2, the heavy capacity group, maximum deceleration of 32.2 feet per second should not be associated with the maximum gross weight of 132,570 pounds. It would be more appropriate to associate the maximum deceleration of vehicles in this classification with the minimum weight and the minimum deceleration with the maximum weight. However, the fact that the distance required to stop increases with an increase in gross weight must be considered.

All vehicles tested did not have decelerations of 32.2 feet per second per second, or 1 g. The deceleration results shown in the tables are sometimes higher than the actual decelerations that would be measured by more sophisticated equipment. Some, but not all, of the vehicles within the different classifications had indicated maximum decelerations of 1 g.; the particularly heavily loaded vehicles had decelerations less than 1 g. The pendulum-type decelerometer used often indicated decelerations of 1 g. when the vehicle bounced, hopped, or jumped during brake application. This was particularly evident when the vehicles tested were carrying relatively light loads in comparison to design loads. The test results do show the relative deceleration performance relations between the different vehicle classifications.

Braking Performance by Vehicle Type

The differences in braking performance attributed to different types of vehicles are shown by the frequency distribution curves in figure 11 for the brake system application and braking distances and deceleration. The curves show the braking performance in percent of vehicles tested by vehicle type, which stopped in a given distance or less, or which reached a deceleration of a given or larger value when simulating an emergency stop from 20 m.p.h. The decelerations measured were not sustained throughout the stops but were the maximum decelerations recorded during the stops. The brake system application and braking distance and the deceleration frequency distributions are evidence that the smaller vehicles are capable of better braking performance.

The improvement in braking performance for different types of commercial vehicles from 1942 to 1963 is shown in figure 12 by the 15th, 50th, and 85th percentile levels (*4*). In general, braking performance has improved during the years in a reduction in the distance required to stop and a decrease in the variability of brake system application and braking distance. This trend in continuing improvement in braking performance was evident in the results of the 1963 brake tests.

The relative effect that different capacity groups and weight groups have on the braking performance of vehicle types is shown by the data in figure 13. The average brake system application and braking distance for each particular grouping was computed and is shown in the figure as a bar of a length in proportion to the respective distance. In normal highway operation, brake system application and braking distance increases with weight for a given type of vehicle and this fact is confirmed by the test data shown (fig. 13).

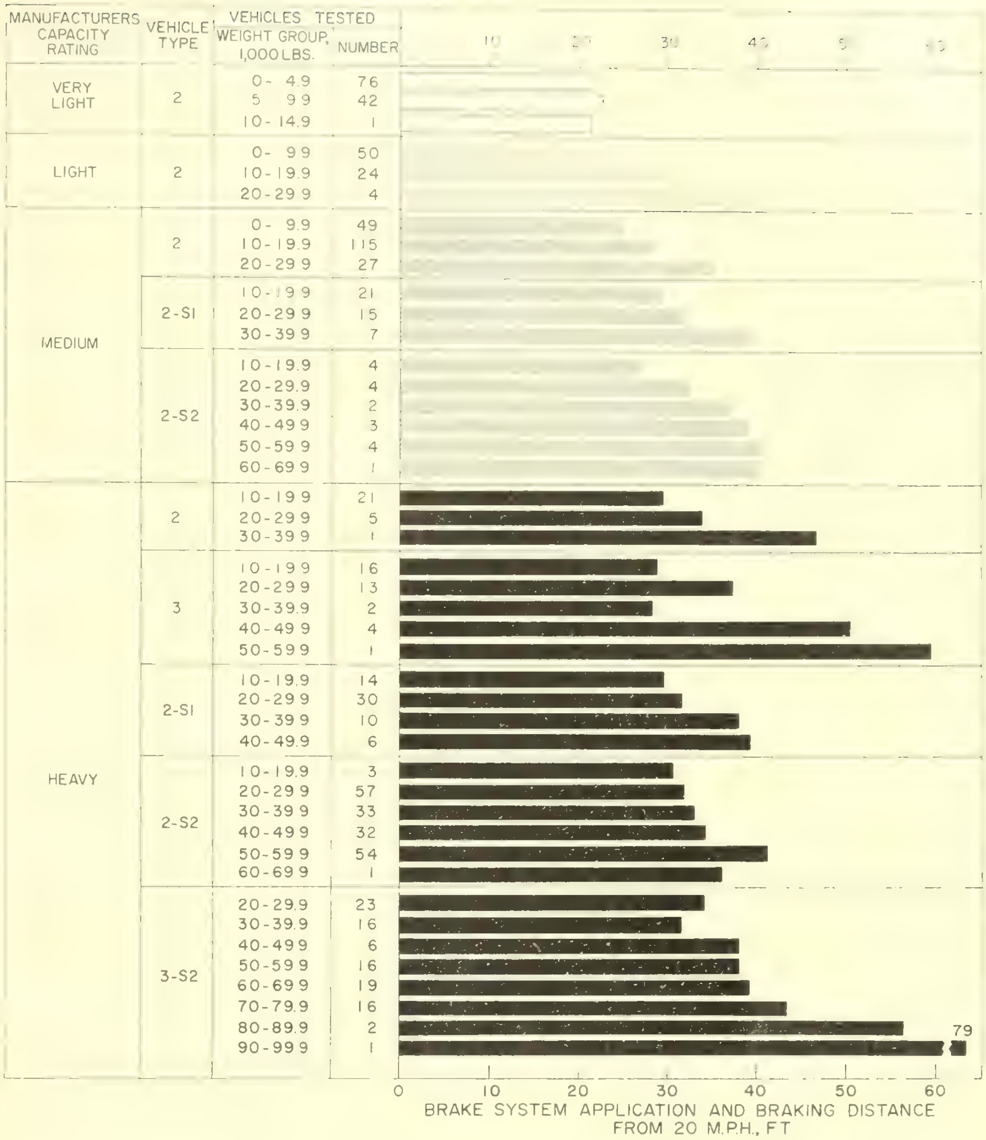

Figure 13.—Braking performance by vehicle types, capacity groups, and weight groups.

Braking Performance in 1955 and 1963

The average weight and the average brake system application and braking distance is given in table 7 by type of vehicle for the vehicles studied in 1955 and 1963. For some types the average weight varied little from 1955 results, but the average weight for others varied considerably (*4*). Part of the variation in average weight can be explained by the chance selection of vehicles to be tested. However, part of the variation in weight also can be attributed to operators of commercial vehicles changing from use of one type of vehicle to another for economic reasons. For example, the 2–S1 vehicles currently are being used to carry lighter loads than previously, although no reduction has been made in the permissible legal weight limits.

The National Committee on Unifrom Traffic Laws and Ordinances specified, in *Uniform Vehicle Code* (*5*), the minimum deceleration and maximum brake system application and braking distances that motor vehicles operating on the highways should obtain when simulating emergency stops from 20 m.p.h. A large percentage of vehicles in the 1963 study met the code requirements; these data are given in table 8. The vehicle types that did not meet the braking requirements of the code were the truck-full trailer and the truck-tractor-semitrailer-full trailer combinations. However, when the brakes on these large vehicle combinations are adjusted properly, they can meet the code requirements. For example, two 3–S3–5 trailer combinations, weighing approximately 133,000 pounds each, were tested. The two truck-tractors were the same make, model, and year, and an air-mechanical brake system was used in each. One trailer combination stopped in 69 feet from 20 m.p.h. and the other stopped in 48 feet, 2 feet less than the code requirement. No maintenance had been

Table 6.—Analysis results for gross vehicle weight, braking system application and braking distance, and deceleration by gross vehicle weight group

Commercial vehicles and capacity groups	Gross vehicle weight group	Test vehicles, GVW and BSABD	Gross vehicle weight				BSABD				Test vehicle	Deceleration (pendulum-type decelerometer)			
			Mean	Standard deviation	Minimum	Maximum	Mean	Standard deviation	Minimum	Maximum		Mean	Standard deviation	Minimum	Maximum
Single-unit trucks:	*1,000 lbs.*	*Number*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Feet*	*Feet*	*Feet*	*Feet*	*Number*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*
2-axle:															
Very light	0-4.9	76	4,053	471	2,545	4,995	21.3	3.9	16	41	76	27.0	3.9	15.1	32.2
	5-9.9	42	5,914	949	5,000	8,700	22.4	3.1	17	29	42	27.3	3.5	20.3	32.2
	10-14.9	1	11,050		11,050	11,050	21.0		21.0	21.0	1	28.3		28.3	28.3
Light	0-9.9	50	7,955	1,192	5,700	9,960	24.5	3.8	8	39	50	27.4	5.0	15.1	32.2
	10-19.9	24	14,037	3,045	10,200	19,110	30.2	5.0	20	45	23	23.7	5.8	14.5	32.2
	20-29.9	4	22,466		20,900	23,840	39.5		32	51	4	15.7		11.9	18.4
Medium	0-9.9	49	8,732	685	6,710	9,790	24.5	4.5	17	40	48	28.0	4.0	17.7	32.2
	10-19.9	115	13,780	2,848	10,050	19,995	28.1	6.3	18	75	111	25.3	4.7	6.4	32.2
	20-29.9	27	22,833	2,297	20,000	27,400	35.3	7.8	26	56	27	19.2	4.8	10.9	29.0
Heavy	10-19.9	21	14,769	2,453	10,605	18,975	29.7	3.9	24	40	20	25.1	4.4	19.3	32.2
	20-29.9	5	23,792		20,185	28,480	34.8		30	44	5	23.2		17.7	30.6
	30-39.9	1	31,410		31,410	31,410	46.0		46.0	46.0	1	14.5		14.5	14.5
All capacity groups	0-9.9	217	6,369	2,110	2,545	9,960	22.9	4.1	16	41	217	27.7	4.1	15.1	32.2
	10-19.9	161	13,930	2,833	10,050	19,995	28.5	6.1	18	75	155	25.1	4.8	6.4	32.2
	20-29.9	36	22,926	2,442	20,000	28,480	35.7	7.5	26	56	35	19.4	5.0	10.9	30.6
	30-39.9	1	31,410		31,410	31,410	46.0		46.0	46.0	1	14.5		14.5	14.5
3-axle:															
Light	20-29.9	1	25,400		25,400	25,400	28.0		28.0	28.0	1	23.2		23.2	23.2
Medium	10-19.9	4	14,332		11,000	17,500	28.2		27	30	4	24.5		22.5	27.4
	20-29.9	1	23,400		23,400	23,400	30.0		30.0	30.0	1	25.8		25.8	25.8
	30-39.9	1	35,950		35,950	35,950	62.0		62.0	62.0	1	11.3		11.3	11.3
Heavy	10-19.9	16	16,389	2,173	12,300	19,300	28.4	4.3	23	41	16	23.6	4.4	17.7	32.2
	20-29.9	13	22,866	2,042	20,100	26,225	36.2	4.1	29	42	13	20.1	4.2	16.1	30.6
	30-39.9	2	37,350		34,800	39,900	28.0		27	29	2	21.7		19.3	24.2
	40-49.9	4	45,651		44,600	47,410	50.5		43	68	4	13.5		8.0	17.7
	50-59.9	1	53,200		53,200	53,200	59.0		59.0	59.0	1	13.5		13.5	13.5
All capacity groups	10-19.9	20	15,978	2,461	11,000	19,300	28.4	4.0	23	41	20	23.8	4.1	17.7	32.2
	20-29.9	15	23,070	2,002	20,100	26,725	35.2	4.6	28	42	15	20.7	4.2	16.1	30.6
	30-39.9	3	36,883	2,675	34,800	39,900	39.3		27	62	3	18.3		8.0	17.7
	40-49.9	4	45,651		44,600	47,410	50.5		43	68	4	13.5		8.0	17.7
	50-59.9	1	53,200		53,200	53,200	59.0		59.0	59.0	1	13.5		13.5	13.5
Truck-tractors with semitrailers:															
2-S1															
Medium	10-19.9	21	17,221	1,824	13,000	19,750	29.7	6.8	24	56	21	28.8	4.1	18.0	32.2
	20-29.9	15	24,752	3,515	20,100	29,510	32.6	4.0	26	40	14	22.9	5.1	17.4	32.2
	30-39.9	7	34,474		32,100	37,000	39.1		30	53	6	19.8		12.2	24.2
Heavy	10-19.9	14	17,978	1,675	14,500	19,890	29.9	6.5	24	48	14	27.0	5.5	14.5	32.2
	20-29.9	30	23,628	2,925	20,105	29,800	32.0	6.4	21	51	29	25.1	5.4	12.6	32.2
	30-39.9	10	33,766	2,334	30,610	37,520	37.8	4.5	33	46	10	18.4	3.9	14.5	22.4
	40-49.9	6	42,358		40,410	45,400	39.2		34	47	6	16.9		12.2	19.3
All capacity groups	10-19.9	35	17,524	1,781	13,000	19,890	29.8	6.6	24	56	35	28.1	4.7	14.5	32.2
	20-29.9	45	24,003	3,139	20,100	29,800	32.2	5.7	21	51	43	24.3	5.4	12.6	32.2
	30-39.9	17	34,057	2,151	30,610	37,520	38.4	6.3	30	53	12	18.9	3.9	12.2	27.4
	40-49.9	6	42,358		40,410	45,400	39.2		34	47	6	16.9		12.2	19.3
2-S2:															
Medium	10-19.9	4	18,050		15,900	19,270	26.3		25	28	4	27.8		22.5	30.6
	20-29.9	4	25,050		23,500	27,130	32.3		25	40	4	26.7		20.9	30.6
	30-39.9	2	35,692		34,985	36,400	36.5		34	39	2	21.3		20.6	21.9
	40-49.9	3	46,473		42,080	49,950	38.3		32	48	3	16.6		12.9	19.3
	50-59.9	4	55,640		53,555	58,345	41.3		34	45	4	15.6		14.5	16.1
	60-69.9	1	63,390		63,390	63,390	41.0		41.0	41.0	1	17.7		17.7	17.7
Heavy	10-19.9	3	19,183		18,500	19,915	31.0		28	33	3	26.0		16.7	32.2
	20-29.9	57	24,201	2,665	20,060	29,830	31.9	6.5	24	66	57	25.9	5.4	9.7	32.2
	30-39.9	33	35,649	3,016	30,140	39,670	34.4	5.3	23	51	32	22.1	4.1	11.9	32.2
	40-49.9	32	44,652	2,509	40,500	48,985	35.3	4.5	27	44	32	21.4	5.1	14.8	32.2
	50-59.9	54	54,888	2,402	50,660	58,800	42.1	8.1	29	67	54	16.5	4.2	8.7	29.3
	60-69.9	1	64,805		64,805	64,805	36.0		36.0	36.0	1	18.4		18.4	18.4
All capacity groups	10-19.9	7	18,536		15,900	19,915	28.3		25	33	7	27.0		16.7	32.2
	20-29.9	61	24,256	2,606	20,060	29,830	31.9	6.5	24	66	61	25.9	5.4	9.7	32.2
	30-39.9	35	35,651	2,931	30,140	39,670	34.5	5.2	23	51	34	22.0	4.0	11.9	32.2
	40-49.9	35	44,808	2,637	40,500	49,950	35.6	4.8	27	48	35	21.1	5.1	12.9	32.2
	50-59.9	58	54,940	2,368	50,600	58,800	42.1	7.9	29	67	58	16.4	4.1	8.7	29.3
	60-69.9	2	64,098		63,390	64,805	38.5		36	41	2	18.0		17.7	18.4

performed on either trailer combination in preparation for the tests. It is almost certain that the trailer combination that stopped in 69 feet could have stopped in a considerably shorter distance if its brakes had been adjusted immediately before the test. It is also possible that a brake adjustment could have improved the braking performance of the other trailer combination.

Axle Loads

Not all vehicle types could be considered in the analyses because either too few vehicles of a given type were tested or weights carried on the principal load-carrying axles varied excessively. Axle loads could be analyzed for only the 2, 2–S1, 2–S2, and 3–S2 types of vehicles. The results from the analyses of the data for the 2, 2–S1, and 3–S2 vehicles were compared with the test results for similar vehicles from previous studies (*4*). Because of large variations in the weights carried on the principal load-carrying axles, previous 2–S2 test results could not be compared with the 1963 study results.

The performance of 2 and 2–S1 vehicles from the brake research studies of 1949, 1955, and 1963 are shown in figure 14. In general, the braking performance of these two types of vehicles improved from one study to the next. The weights on the steering axles were not considered in the data shown. For the type 2, single-unit vehicles, the rear axles were grouped in weight increments of 4,000 pounds and the braking performance was then computed for the groups and plotted at the midpoint of the weight group. The same analysis procedure was used for the 2–S1 vehicles, however, data were considered only for those trailer combinations for which the weights of the truck-tractor drive axle and the trailer axle were in the same 4,000-pound group.

Primarily because of difficulty encountered in establishing weight increments in which a sufficient number of observations could be obtained for 2–S2 of trailer combinations, the data were treated differently. The method of least squares was used to compute the linear regression equation that best fit the data. In the analysis of the data for 2–S2 trailer combinations information was used only on trailer combinations for which the truck-tractor drive axle weight equaled or exceeded 16,000 pounds. In the analysis of the data on the 3–S2 combinations, test results were used only for trailer combinations in which both sets of tandem axles were within 4,000 pounds of each other.

The braking performance for the 2–S2 and 3–S2 trailer combinations in relation to the weight on the tandem axles is shown in figure 15. The regression curve determined for the 2–S2 trailer combinations is approximately parallel to and 2 feet below the curve for the 3–S2 trailer combinations. The coefficient of

Table 6.—Analysis results for gross vehicle weight, braking system application and braking distance, and deceleration by gross vehicle weight group—Continued

Commercial vehicles and capacity groups	Gross vehicle weight group	Test vehicles, GVW and BSABD	Gross vehicle weight				BSABD				Test vehicle	Deceleration (pendulum-type decelerometer)			
			Mean	Standard deviation	Minimum	Maximum	Mean	Standard deviation	Minimum	Maximum		Mean	Standard deviation	Minimum	Maximum
	1,000 lbs.	*Number*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Feet*	*Feet*	*Feet*	*Feet*	*Number*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Ft./sec./sec.*
Truck-tractors with semitrailers:															
3–S2:															
Medium	70–79.9	1	72,480		72,480	72,480	68.0		68	68	1	9.7		9.7	9.7
Heavy	20–29.9	23	26,804	2,385	23,100	29,755	34.4	6.0	25	50	22	27.6	6.4	13.9	32.2
	30–39.9	16	33,575	2,849	30,230	39,700	31.7	4.5	24	39	16	27.8	4.6	17.7	32.2
	40–49.9	6	42,112		40,400	44,540	37.7		33	42	6	22.0		16.1	32.2
	50–59.9	16	56,039	2,953	50,900	59,350	37.7	7.8	25	50	16	20.9	4.7	13.8	28.0
	60–69.9	19	64,709	2,947	60,090	69,200	38.7	4.6	26	44	19	17.8	3.7	12.9	25.1
	70–79.9	16	73,110	3,113	70,300	79,680	43.9	9.5	32	60	16	15.5	4.0	9.7	22.5
	80–89.9	2	80,495		80,890	80,590	56.0		38	74	2	15.1		9.7	20.6
	90–99.9	1	94,650		94,650	94,650	79.0		79	79	1	8.7		8.7	8.7
All capacity groups	20–29.9	23	26,804	2,385	23,100	29,755	34.4	6.0	25	50	22	27.6	6.4	13.8	32.2
	30–39.9	16	33,575	2,849	30,230	39,700	31.7	4.5	24	39	16	27.8	4.6	17.7	32.2
	40–49.9	6	42,112		40,400	44,540	37.7		33	42	6	22.0		16.1	32.2
	50–59.9	16	56,039	2,953	50,900	59,350	37.7	7.8	25	50	16	20.9	4.7	13.8	28.0
	60–69.9	19	64,709	2,947	60,090	69,200	38.7	4.6	26	44	19	17.8	3.7	12.9	25.1
	70–79.9	17	73,073	3,018	70,300	79,680	45.4	10.9	32	68	17	15.2	4.2	9.7	22.5
	80–89.9	2	80,495		80,400	80,590	56.0		38	74	2	15.1		9.7	20.6
	90–99.9	1	94,650		94,650	94,650	79.0		79	79	1	8.7		8.7	8.7
Trucks with full trailers:															
3–2:															
Heavy	20–29.9	11	25,582	2,548	22,400	29,600	33.9	4.4	23	40	11	31.2	2.8	24.2	32.2
	30–39.9	2	36,800		36,400	37,200	33.5		33	34	2	29.0		24.2	32.2
	40–49.9	2	45,000		40,400	49,600	43.0		36	50	2	16.3		12.9	19.6
	50–59.9	0									0				
	60–69.9	1	67,290		67,290	67,290	57.0		57	57	1	11.3		11.3	11.3
	70–79.9	10	76,090	1,066	74,900	78,200	49.2	6.3	43	60	10	15.6	5.5	11.3	30.6
Truck-tractors with semitrailers and full trailers:															
2–S1–2:															
Heavy	20–29.9	7	26,313		24,500	27,700	35.7		32		7	29.7		22.5	32.2
	30–39.9	8	32,275		30,200	36,600	34.5		27	43	8	26.2		20.9	32.2
	40–49.9	0									0				
	50–59.9	1	51,200		51,200	51,200	44.0		44	44	1	17.7		17.7	17.7
	60–69.9	3	62,900		60,800	66,300	49.0		44	52	3	17.0		14.5	19.3
	70–79.9	27	76,284	1,971	70,300	79,000	52.7	10.8	40	75	27	14.9	2.9	9.3	20.9
	80–89.9	2	82,235		81,700	82,770	47.5		45	50	2	14.0		11.9	16.1
Trucks with full trailers and truck-tractors with semitrailers and full trailers:															
Heavy	20–29.9	19	25,784	2,054	22,400	29,600	34.6	4.2	23	48	19	30.2	3.6	22.5	32.2
	30–39.9	14	34,232	2,954	30,200	39,030	34.6	4.6	27	40	14	27.3	3.1	20.9	32.2
	40–49.9	2	45,000		40,400	49,600	43.0		36	50	2	16.3		12.9	19.6
	50–59.9	2	51,700		51,200	52,200	37.5		31	44	2	17.5		17.4	17.7
	60–69.9	5	63,454		60,800	67,290	50.4		44	57	5	15.0		11.3	19.3
	70–79.9	38	76,173	1,774	70,300	79,000	51.7	9.7	40	75	38	15.2	3.6	10.3	30.6
	80–89.9	2	82,235		81,700	82,770	47.5		45	50	2	14.0		11.9	16.1
	90–99.9	3	95,153		94,150	96,260	56.0		49	64	3	15.4		10.6	17.7
	100–109.9	1	108,120		108,120	108,120	38.0		38	38	1	20.3		20.3	20.3
	110–119.9	1	113,460		113,460	113,460	38.0		38	38	1	20.9		20.9	20.9
	120–129.9	0									0				
	130–139.9	2	132,535		132,500	132,570	58.5	14.8	48	69	2	13.8		11.6	16.1

Table 7.—Average weight and brake system application and braking distance for commercial vehicles tested in 1955 and 1963

Commercial vehicles	1955			1963		
	Vehicles	Average weight	Average BSABD from 20 m.p.h.	Vehicles	Average weight	Average BSABD from 20 m.p.h.
	Number	*Pounds*	*Feet*	*Number*	*Pounds*	*Feet*
Single-unit trucks:						
2-axle, very light	107	5,200	24	119	4,740	22
2-axle, other than very light	293	14,200	31	296	13,100	28
3-axle	73	28,400	39	43	23,500	34
Truck-tractors with semitrailers:						
2–S1	129	32,100	40	103	24,500	[illegible]
2–S2	153	40,400	42	199	39,000	[illegible]
2–S3				2	32,600	[illegible]
3–S2	66	53,700	46	100	50,300	[illegible]
Trucks with full trailers:						
2–2	16	45,900	51	[illegible]	42,800	[illegible]
3–2	46	63,900	54	[illegible]	49,000	41
Truck-tractors with semitrailers and full trailers:						
2–S1–2	44	59,700	56	49	59,800	47
2–S2–2	7	62,200	54	5	75,400	50
2–S2–3	2	52,000	41	4	88,800	[illegible]
3–S1–2	1	78,600	43	1	52,200	[illegible]
3–S2–2				2	37,000	57
3–S3–5				2	[illegible]	58

correlation for the 2–S2 and 3–S2 trailer combinations of 0.41 and 0.60, respectively, indicates that the regression curves did not fit the data as well as might be hoped for. A large amount of scatter about the regression line, caused by a large variation in the brake system application and braking distance, was responsible for the small coefficients. The coefficients of determination indicate that 17 and 36 percent of the total variation in brake system application and braking distance for the 2–S2 and 3–S2 trailer combinations, respectively, can be attributed to the tandem axle weights and the remaining or unexplained variation must be attributed to other factors. Such factors include inadequate brake system maintenance and/or poor brake adjustment. The linear regression curve for the 3–S2 trailer combinations tested in 1955 is also shown in figure 15. This curve indicates that the braking performance in relation to tandem axle loadings was poorer in the 1955 study than in the 1963 study. A larger percentage of the variation in brake system application and braking distance in the 1955 study could be explained by tandem-axle weight.

Before the braking performance in relation to the manufacturers gross vehicle or combination weight rating could be evaluated, the manufacturers weight rating for the test vehicle had to be determined. The manufacturers gross vehicle weight ratings were used to evaluate the braking performance of single-unit trucks and the manufacturers gross combination weight ratings were used to evaluate braking performance of trailer combinations. Usually the weight rating appears on the manufacturers identification

Table 8.—Braking test results for 1955 and 1963 compared with *Uniform Vehicle Code* requirements

Commercial vehicles	Deceleration			BSABD		
	UVC requirements	Vehicles within requirements		UVC requirements	Vehicles within requirements	
		1955	1963		1955	1963
	Ft./sec./sec.	*Percent*	*Percent*	*Feet*	*Percent*	*Percent*
Single-unit trucks:						
2-axle, very light	14	100	100	30	84	97
2-axle, other than very light	14	94	98	40	84	95
3-axle	14	85	91	40	53	75
Truck-tractors with semitrailers:						
2-S1	14	83	97	50	81	97
2-S2	14	82	91	50	80	94
3-S2	14	76	89	50	64	92
Trucks with full trailers	14	51	80	50	38	86
Truck-tractors with semitrailers and full trailers	14	69	79	50	41	71

Table 9.—Mean, standard deviation, and minimum ratios of GVW to manufacturers weight rating

Commercial vehicles (all capacity groups)	Gross vehicle weight, 1,000 lbs.	Number	Ratio, $\frac{\text{gross vehicle weight}}{\text{manufacturers weight rating}}$			
			Mean	Standard deviation	Minimum	Maximum
Single-unit trucks:						
2-axle	0–9.9	217	0.65	0.192	0.37	1.12
	10–19.9	161	0.71	0.196	0.37	1.26
	20–29.9	36	1.11	0.208	0.65	1.49
	30–39.9	1	1.05	---	1.05	1.05
3-axle	10–19.9	20	0.52	0.146	0.36	0.92
	20–29.9	15	0.73	0.316	0.45	1.59
	30–39.9	3	1.19	------	0.93	1.63
	40–49.9	4	1.15		1.06	1.36
Truck-tractors with semitrailers:						
2-S1	10–19.9	35	0.40	0.067	0.31	0.56
	20–29.9	45	0.51	0.161	0.27	0.81
	30–39.9	16	0.71	0.128	0.48	1.00
	40–49.9	6	0.80	---	0.63	0.90
2-S2	10–19.9	7	0.43		0.34	0.64
	20–29.9	62	0.47	0.130	0.30	0.97
	30–39.9	34	0.67	0.105	0.46	0.87
	40–49.9	35	0.81	0.200	0.53	1.45
	50–59.9	58	1.02	0.158	0.70	1.41
	60–69.9	2	1.25		1.08	1.41
3-S2	20–29.9	22	0.42	0.055	0.31	0.54
	30–39.9	14	0.51	0.063	0.43	0.61
	40–49.9	4	0.68		0.58	0.84
	50–59.9	11	0.81	0.084	0.69	0.93
	60–69.9	17	0.94	0.114	0.80	1.15
	70–79.9	13	1.09	0.164	0.90	1.45
	80–89.9	1	1.24		1.24	1.24

Table 10.—Braking performance of trailer combinations with and without brakes on the steering axle

Commercial vehicles	Brakes on steering axle				No brakes on steering axle			
	Vehicles tested	Average weight	Average deceleration	Average BSABD	Vehicles tested	Average weight	Average deceleration	Average BSABD
Truck-tractors with semitrailers:	*No.*	*Lbs.*	*Ft./sec./sec.*	*Ft.*	*No.*	*Lbs.*	*Ft./sec./sec.*	*Ft.*
3-S2	60	49,800	23	37	40	51,000	21	40
Truck-tractors with semitrailers and full trailers:								
2-S1-2	28	56,400	18	46	21	64,400	19	48
3-S2-2	1	39,000	32	35	1	35,000	27	39

plate attached to the vehicle; however, often the manufacturers specifications had to be consulted. When the weight rating had been determined, a ratio was computed between the gross vehicle weight and the manufacturers weight rating for test vehicles. Sometimes the ratio could not be computed because the manufacturers weight rating could not be found on the vehicle or determined from the vehicle specifications; data for these vehicles were not used in the analysis.

The analysis of braking performance ratio by GVW and manufacturers weight rating was made for the 2- and 3-axle, single-unit trucks and for the 2–S1, 2–S2, and 3–S2 truck-tractor-semitrailer combinations. Because the manufacturers gross combination weight rating for many of the multicombination vehicles could not be determined, data for these vehicles were not included in the analysis. Results of the analysis of braking performance in relation to the ratio of gross weight and manufacturers weight ratings are shown in table 9 by vehicle type and weight group.

The effect of an increase in the gross weight to the manufacturers weight rating on braking performance is shown in figure 16. As the ratio of the gross vehicle weight to manufacturers weight rating increased, the brake system application and braking distance also increased but the peak deceleration decreased. Mean values for ratio, deceleration, and distance are plotted in figure 16 at the mean weight for the different test weight groups. All trailer combinations, except the 2–S1, had gross vehicle weights of more than the recommended manufacturers rating; this is indicated by a ratio of more than 1. With one exception, when the ratio was less than 1, the vehicles met the *Uniform Vehicle Code* (*5*) recommendations for braking performance: the 3-axle, single-unit trucks required approximately 2 feet more than the recommended distance of 40 feet from a speed of 20 m.p.h. In an evaluation of the braking performance of the types of commercial vehicles tested in this research, the fact must be recognized that braking systems can be designed to meet given performance requirements provided that the gross vehicle weight does not exceed the manufacturers suggested weight rating and that the braking systems are properly maintained.

Table 11.—Braking performance with all axles braked and without steering axles braked, in test by Committee on Winter Driving Hazards

Commercial vehicles	Weight	BSABD from 20 m.p.h.	
		All axles braked	Steering axles not braked
	Pounds	*Feet*	*Feet*
3-S2	24,830	24	30
3-2	22,300	21	25
2-S1-2	22,090	26	31

No Brakes on Steering Axle

Some States and the Interstate Commerce Commission permit, in their motor-vehicle regulations, certain vehicles to operate without any brakes on the steering axle. In the 1963 braking performance test, combination vehicles were tested that did not have front wheel brakes; these are listed in table 10. Except for the 3–S2 trailer combinations a large difference existed in the mean gross weights between the trailer combinations that did and those that did not have brakes on the steering axle. Consequently, the longer distance required for stopping by the combinations without front wheel brakes cannot be attributed entirely to the fact that one axle was not braked—the poorer performance also could have been attributed partially to the weight differential. The additional distance was approximately 2 to 4 feet.

In 1958 the National Safety Council's Committee on Winter Driving Hazards conducted tests on dry pavement for empty, combination vehicles, both with and without the steering axles braked (6). The findings in terms of the brake system application and braking distance when making emergency-type stops from 20 m.p.h. for both braking conditions are shown in table 11. The brake system application and braking distance increased 4 to 6 feet, when the steering axle was not braked.

Confidence Intervals

The commercial vehicles tested were grouped according to type, capacity, brake system, and weight. Similar commercial vehicles were classified into groups, and then considered as samples from the group populations. The standard errors of the means were computed for the groups that had at least 10 observations. Confidence intervals then were computed for each commercial vehicle group having 10 or more observations. By using the confidence interval, the levels of braking performance for each individual group could be estimated and the degree of reliability of estimates known. For each group, the 95 percent confidence intervals for the means of the gross weight, deceleration, and brake system application and braking distance were determined; their confidence intervals were computed in the same manner as those for the passenger cars. The confidence intervals by type of brake systems and by weight group are shown in table 12.

Findings of Analyses

Passenger cars

The following findings concerning passenger cars were obtained from analyses of the 1963 test data.

- The average weights of foreign cars, compact cars, and standard size cars differed significantly from each other at the 0.05 level.
- Little change has occurred since 1955 in the deceleration performance of all passenger cars, when considered as a group. Comparison of decelerations of the foreign cars with the compact cars, however, showed that the compact cars had significantly larger average decelerations in the 1963 tests at the 0.05 level.
- Some decrease since 1955 in the brake system application and braking distance was shown in the 1963 test results, particularly above the 50th percentile level. In the comparison of the average brake system application and braking distances for the different passenger car classifications studied in 1963, only results of the compact car comparison with the standard size car differed significantly at the 0.05 level.
- The variability in the brake system application and braking distances has continued to decrease since 1955.
- The mean brake system application and braking distances for the different test years were significantly different at the 0.01 level.
- According to 1963 test results, 95 percent of the time the mean brake system application and braking distances for the passenger car classifications can be expected to be within the following distance intervals: foreign, 18.4 to 20.2 feet; compact, 18.7 to 19.3 feet; and standard size, 19.8 to 20.2 feet.

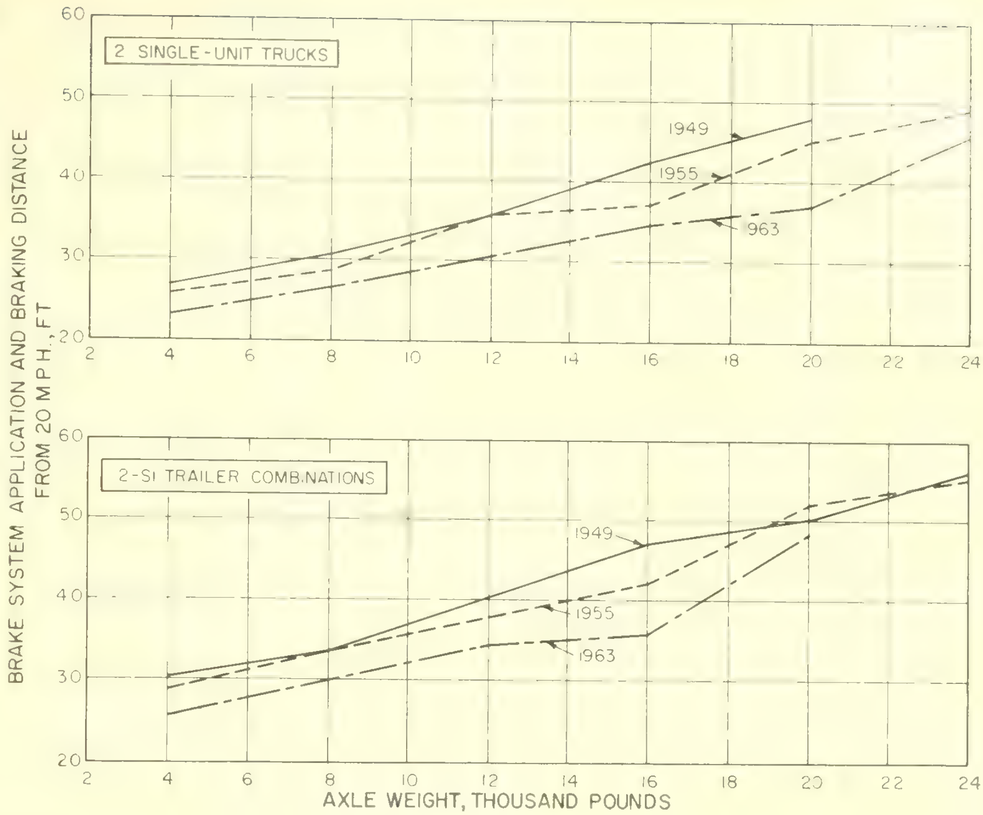

Figure 14.—Brake system application and braking distances by vehicle axle weights, by test years.

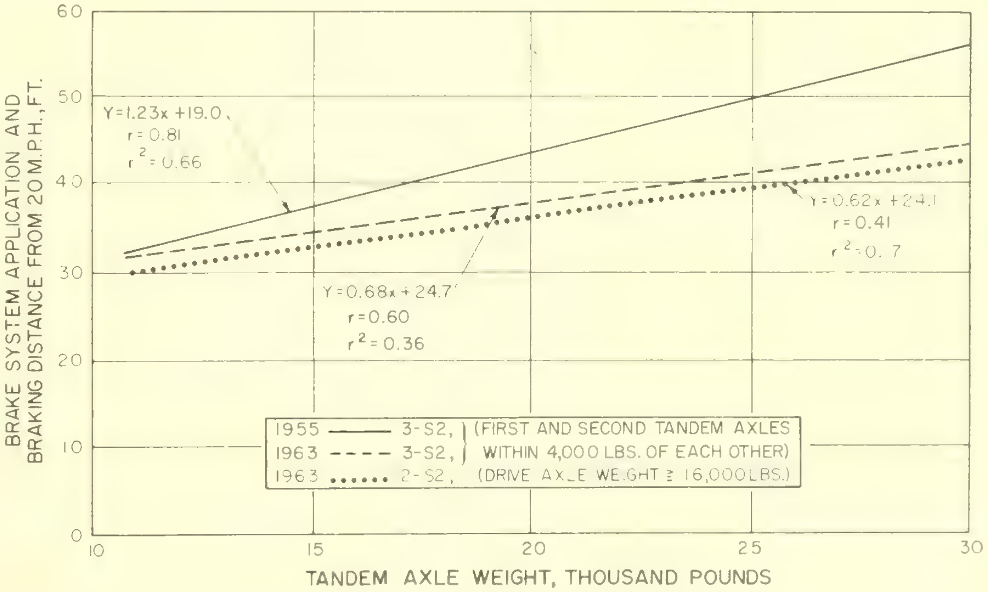

Figure 15.—Brake system application and braking distances by tandem axle weights for 2–S2 and 3–S2 vehicles.

Commercial vehicles

The following findings were obtained from the analyses of the 1963 test results for commercial vehicles.

- The average brake system application and braking distance since 1955 has decreased 2 to 3 feet for the very light 2-axle trucks to 10 feet or more for some of the heavier trailer combinations. Since the 1955 tests all the commercial vehicles had improved deceleration performance from approximately 5 percent for very light 2-axle trucks to 15 percent for heavier multitrailer combinations.
- The variability in the brake system application and braking distance for similar types of vehicles continued to decrease.
- The brake system application and braking distance generally has decreased since 1955 regardless of the vehicle type, weight group, or manufacturers capacity group.
- In the 1963 tests, a larger percentage than in the 1955 tests of commercial vehicles could meet the brake system application and braking distance and deceleration requirements recommended in the *Uniform Vehicle Code.*

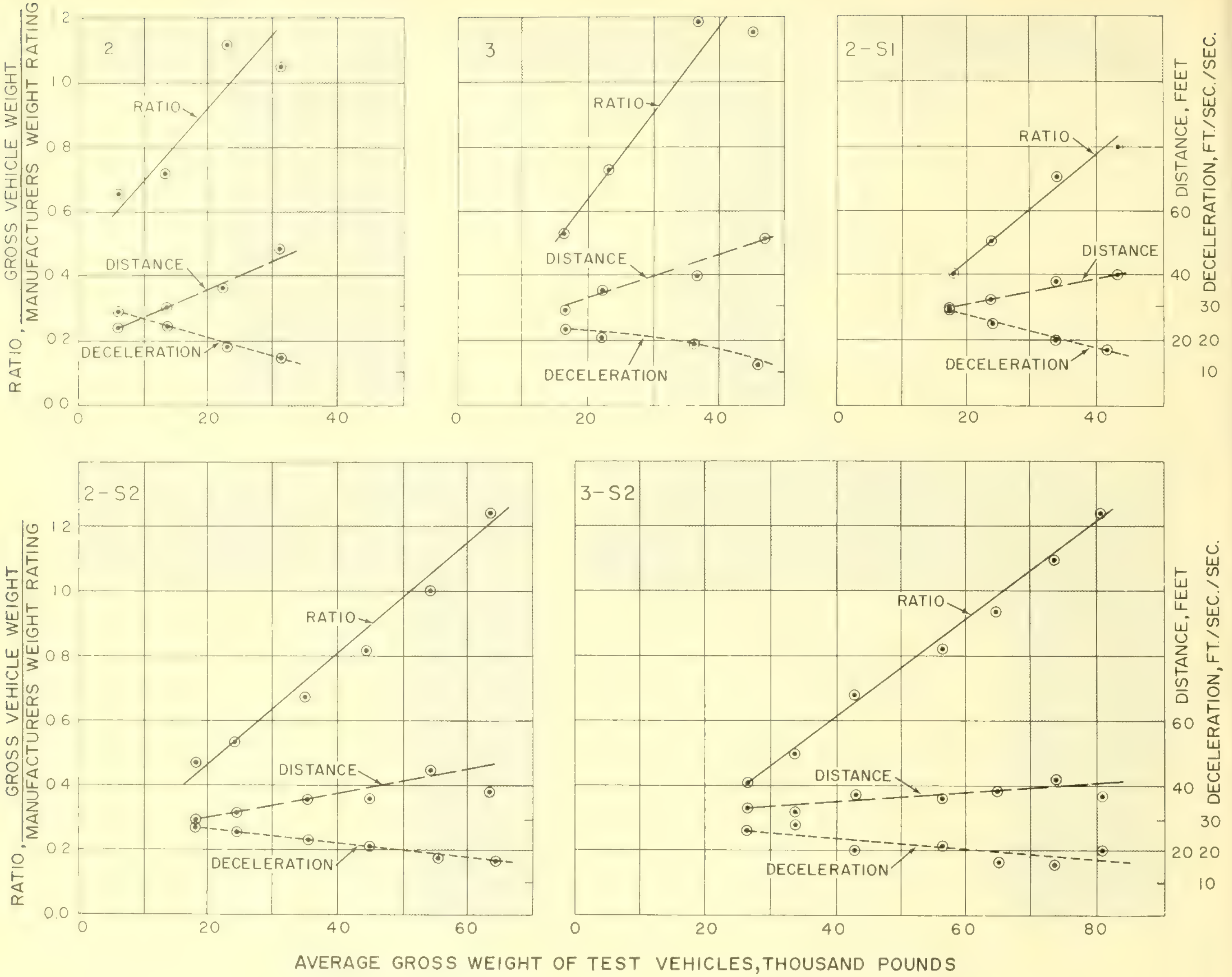

Figure 16.—Relation of GVW to manufacturers weight rating, distance, deceleration, and weight.

- Results of the axle load analysis showed that the brake system application and braking distances for similar axle loadings decreased approximately 3 feet from 1955 to 1963 for both 2-axle, single-unit trucks and 2–S1 trailer combinations.
- The relation between the distance required to stop and tandem axle weights of 2–S2 and 3–S2 trailer combinations could not be clearly defined in the analysis of test results. Only a small part of the total variation in the brake system application and braking distance could be explained by axle loading.
- When the ratio of gross vehicle weight to the manufacturers weight rating was less than 1, the vehicles met the *Uniform Vehicle Code* recommendations. However, the 3-axle, single-unit trucks required approximately 42 feet rather than the 40 feet to stop at 20 m.p.h.
- The 1963 test results and the National Safety Council Committee on Winter Driving Hazards studies showed that the brake system application and braking distance is several feet longer when the steering axle is not braked than when it is.
- According to the 1963 test results, the mean brake system application and braking distance for all commercial vehicles of a given type can be expected 95 percent of the time to be within the following distance intervals: 2-axle, single-unit trucks, 26 to 27 feet; 3-axle, single-unit trucks, 31 to 37 feet; 2–S1, 31 to 34 feet; 2–S2, 35 to 37 feet; 3–S2, 36 to 40 feet; 3–2, 38 to 45 feet; and 2–S1–2, 43 to 50 feet.

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(*7*) *The 1958 Winter Test Report,* by Committee on Winter Driving Hazards, National Safety Council, May 1962.

Table 12.—Confidence interval for commercial vehicles according to brake system and gross vehicle weight groups

Commercial vehicles and capacity groups	Brake systems [1]	Standard error of mean			95 percent confidence intervals						Gross vehicle weight groups	Standard error of mean			95 percent confidence intervals					
		Gross weight	Deceleration [2]	BSABD	Gross weight		Deceleration		BSABD			Gross weight	Deceleration [2]	BSABD	Gross weight		Deceleration [2]		BSABD	
					From	To	From	To	From	To					From	To	From	To	From	To
Single-unit trucks:	*Types*	*Pounds*	*Ft./sec./sec.*	*Feet*	*Pounds*	*Pounds*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Feet*	*Feet*	*1,000 lbs.*	*Pounds*	*Ft./sec./sec.*	*Feet*	*Pounds*	*Pounds*	*Ft./sec./sec.*	*Ft./sec./sec.*	*Feet*	*Feet*
2-axle:																				
Very light	H	103.1	0.35	0.34	4,480	4,890	26.9	28.3	20.9	22.3	0-4.9	54.0	0.45	0.45	3,950	4,160	27.0	28.8	20.4	22.2
											5-9.9	146.5	0.54	0.48	5,630	6,200	26.2	28.4	21.5	23.3
	All	115.4	0.35	0.33	4,540	4,990	27.0	28.4	21.0	22.2										
Light	H	1,113.4	1.38	1.95	7,940	12,310	19.2	24.6	25.6	33.2	0-9.9	168.6	0.71	0.54	7,620	8,280	26.0	28.8	23.4	25.6
	VBH	558.3	0.72	0.73	9,570	11,760	25.1	27.9	25.0	27.8	10-19.9	621.4	1.21	1.22	12,820	15,250	21.3	26.1	27.8	32.6
	All	497.4	0.67	0.70	9,600	11,550	24.4	27.0	25.6	28.4										
Medium	VBH	367.7	0.38	0.42	12,910	14,360	24.8	26.2	26.6	28.2	0-9.9	97.9	0.58	0.64	8,540	8,920	26.9	29.1	23.3	25.8
	AM	2,019.6	2.38	5.32	13,590	21,500	14.1	23.5	27.4	48.2	10-19.9	265.7	0.45	0.59	13,260	14,300	24.4	26.2	26.9	29.3
											20-29.9	441.7	0.92	1.50	21,970	23,700	17.4	21.0	32.4	38.2
	All	354.0	0.38	0.50	13,070	14,460	24.4	25.8	27.1	29.1										
Heavy	VBH	1,594.0	1.42	1.74	12,650	18,900	24.8	30.4	27.0	33.8	10-19.9	535.6	0.98	0.85	13,720	15,820	23.2	27.0	28.0	31.4
	AM	1,312.3	0.91	1.35	15,360	20,500	20.2	23.8	29.1	34.3										
	All	1,014.6	0.96	1.06	15,070	19,040	22.4	26.2	29.1	33.3										
All capacity groups	H	237.2	0.37	0.42	5,110	6,040	26.2	27.6	21.9	23.5	0-9.9	143.2	0.28	0.28	6,090	6,650	27.2	28.3	22.4	23.5
	VBH	313.2	0.33	0.36	12,280	13,510	25.3	26.5	26.5	27.9	10-19.9	223.2	0.39	0.48	13,490	14,370	24.3	25.9	27.6	29.4
	AM	1,095.9	1.11	2.27	15,640	19,930	18.5	22.9	29.6	38.4	20-29.9	407.0	0.84	1.25	22,130	23,720	17.8	21.1	33.3	38.2
	All	285.6	0.25	0.33	10,240	11,360	25.4	26.4	25.6	26.8										
3-axle:																				
Heavy	AM	1,996.4	0.83	1.72	21,650	29,480	18.2	21.4	32.5	39.3	10-19.9	543.3	1.10	1.08	15,320	17,450	21.4	25.8	26.3	30.5
											20-29.9	565.7	1.16	1.14	21,760	23,970	17.8	22.4	34.0	38.4
	All	1,818.2	0.88	1.61	20,600	27,730	19.1	22.5	31.3	37.7										
All capacity groups	VBH	1,488.0	1.18	0.78	13,690	19,520	23.5	28.1	25.5	28.5	10-19.9	550.6	0.92	0.89	14,900	17,060	22.0	25.6	26.7	30.1
	AM	1,918.8	0.84	1.82	21,870	29,400	18.1	21.3	32.9	40.1	20-29.9	517.3	1.09	1.19	22,060	24,080	18.6	22.8	32.9	37.5
	All	1,613.7	0.80	1.53	20,370	26,700	19.5	22.7	31.3	37.3										
Truck-tractors with semitrailers:																				
2-S1:																				
Medium	VBH-VM	1,769.6	1.34	1.93	19,130	26,070	21.7	26.9	30.2	37.8	10-19.9	398.3	0.90	1.48	16,440	18,000	27.0	30.6	26.8	32.6
	AM-AM	1,377.7	1.35	1.33	20,120	25,520	24.2	29.4	28.5	33.7	20-29.9	908.3	1.36	1.03	22,970	26,530	20.2	25.6	30.6	34.6
	All	1,032.3	0.88	1.07	20,630	24,680	23.7	27.1	30.1	34.3										
Heavy	AM-AM	1,043.6	0.85	0.94	23,700	27,790	21.9	25.3	31.4	35.0	10-19.9	447.9	1.47	1.74	17,100	18,860	24.1	29.9	26.5	33.3
											20-29.9	533.8	1.00	1.17	22,580	24,670	23.1	27.1	29.7	34.3
											30-39.9	738.6	1.23	1.42	32,320	35,210	16.0	20.8	35.0	40.6
	All	1,012.9	0.80	0.87	23,890	27,860	21.9	25.1	31.5	34.9										
All capacity groups	VBH-VM	1,990.6	1.28	1.76	21,020	28,820	20.8	25.8	31.5	38.3	10-19.9	300.8	0.79	1.11	16,930	18,110	26.6	29.7	27.6	32.0
	AM-AM	852.0	0.73	0.77	23,240	26,580	23.0	25.8	31.1	34.1	20-29.9	467.8	0.82	0.85	23,090	24,920	22.7	25.9	30.5	33.9
											30-39.9	522.1	0.98	1.53	33,030	35,080	17.0	20.8	35.4	41.4
	All	744.2	0.60	0.67	23,070	25,990	23.1	25.5	31.5	34.1										
2-S2:																				
Medium	AM-AM	4,435.3	1.65	1.92	29,720	47,100	19.0	25.4	30.4	38.0										
	All	3,805.2	1.43	1.77	29,720	44,630	18.9	24.5	31.3	38.3										
Heavy	AM-AM	975.1	0.45	0.57	37,560	41,380	20.5	22.3	35.0	37.2	20-29.9	353.0	0.72	0.86	23,510	24,890	24.5	27.3	30.2	33.6
											30-39.9	525.4	0.72	0.92	34,620	36,680	20.7	23.5	32.6	36.2
											40-49.9	443.3	0.90	0.80	43,780	45,520	19.6	23.2	33.7	36.9
											50-59.9	326.8	0.57	1.10	54,250	55,530	15.4	17.6	39.9	44.3
	All	973.1	0.45	0.57	37,380	41,190	20.6	22.4	34.9	37.1										
All capacity groups	AM-AM	956.1	0.44	0.55	37,520	41,270	20.6	22.4	34.9	37.1	20-29.9	333.7	0.69	0.83	23,600	24,910	24.6	27.3	30.3	33.5
											30-39.9	495.1	0.69	0.88	34,680	36,620	20.7	23.4	32.8	36.2
											40-49.9	445.4	0.86	0.81	43,930	45,680	19.4	22.8	34.0	37.2
											50-59.9	310.8	0.54	1.04	54,330	55,550	15.3	17.5	40.1	44.1
	All	947.7	0.43	0.55	37,230	40,950	20.7	22.3	34.8	37.0										
3-S2:																				
Heavy	AM-AM	1,895.9	0.70	0.94	46,630	54,060	20.3	23.1	36.1	39.7	20-29.9	496.9	1.36	1.25	25,830	27,780	24.9	30.3	32.0	36.9
											30-39.9	712.3	1.15	1.13	32,180	34,970	25.6	30.1	29.5	33.9
											50-59.9	738.3	1.18	1.95	54,590	57,490	18.6	23.2	33.9	41.5
											60-69.9	675.9	0.85	1.06	63,380	66,030	16.1	19.5	36.6	40.8
											70-79.9	778.3	1.00	2.38	71,580	74,630	13.5	17.5	39.2	48.6
	All	1,895.3	0.71	0.93	46,360	53,790	20.4	23.2	36.1	39.7										
All capacity groups											20-29.9	496.9	1.36	1.25	25,830	27,780	24.9	30.3	32.0	36.9
											30-39.9	712.3	1.15	1.13	32,180	34,970	25.6	30.1	29.5	33.9
											50-59.9	738.3	1.18	1.95	54,590	57,490	18.6	23.2	33.9	41.5
											60-69.9	675.9	0.85	1.06	63,380	66,030	16.1	19.5	36.6	40.8
	AM-AM	1,889.9	0.71	0.97	46,860	54,270	20.2	23.0	36.5	40.1	70-79.9	732.5	1.02	2.65	71,640	74,510	13.2	17.2	40.2	50.6
	All	1,889.6	0.71	0.97	46,600	54,010	20.3	23.1	36.3	40.1										
Trucks with full trailers:																				
3-2:																				
Heavy	AM-AM	4,66[illegible].3	1.75	1.86	39,820	58,110	19.7	26.5	37.7	44.9	20-29.9	767.5	0.84	1.33	24,080	27,090	29.6	32.9	31.3	36.5
											70-79.9	337.3	1.74	1.99	75,430	76,750	12.2	19.0	45.3	53.1
Truck-tractors with semitrailers and full trailers:																				
2-S1-2:																				
Heavy	AM-AM-AM.	3,156.0	0.98	1.69	54,360	66,740	17.2	21.0	43.2	49.8	70-79.9	379.0	0.56	2.08	75,540	77,030	13.8	16.0	48.6	56.8
All capacity groups.	All	3,173.0	0.96	1.66	53,620	66,060	17.2	21.0	43.2	49.8										
All other than 2-S1-2:																				
Heavy	AM-AM-AM.	10,236.0	1.81	3.40	60,380	100,500	16.8	23.8	38.1	51.5										
All capacity groups.	AM-AM-AM.	9,494.9	1.76	3.21	61,620	98,840	16.4	23.2	39.1	51.7										
Trucks with full trailers and truck-tractors with semitrailers and full trailers:																				
Heavy											20-29.9	471.1	0.83	0.96	24,860	26,710	28.6	31.8	32.7	36.5
											30-39.9	789.8	1.04	1.23	32,680	35,780	25.3	29.3	32.2	37.0
											70-79.9	288.0	0.58	1.57	75,610	76,740	14.1	16.3	48.6	54.8

[1] All refers to total number of vehicles tested in each category, as identified in table 5.

[2] Measured by pendulum type decelerometer.

Relations of Gross Weights and Horsepowers of Commercial Vehicles

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

**Reported by [1] JOHN M. WRIGHT and SAMUEL C. TIGNOR,
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Results of a study on the relation between gross weight and net engine horsepower of commercial vehicles are presented in this article. Data from the braking performance study were used to update current information on weight-power ratios of trucks and to investigate the trend in these ratios since 1949. A total of 1,026 commercial vehicles, in a large variety of types, weights, and horsepowers were sampled in three States from routes having a heavy concentration of commercial vehicles.

Data from the study were used to determine the effect of weight-power ratio requirements on the trucking industry and to determine the percentage of vehicles affected by a minimum performance requirement. The data collected in the study indicate that the dissimilarity in performance of passenger cars and commercial vehicles is lessening. There is a trend toward decreasing weight-power ratios and a performance requirement of 400 pounds per horsepower would affect only a small percentage of the commercial vehicles. However, a substantial reduction in the weight-power ratio is still necessary to put passenger and commercial vehicles on similar performance levels.

Introduction

THE BUREAU of Public Roads periodic braking performance study made in 1963 also provided information for determining the ratios of commercial vehicle gross weight to net engine horsepower. An engine net horsepower rating for each truck or trailer combination tested was also recorded. Field observations were made in Maryland, Michigan, and California on routes having a heavy concentration of commercial vehicles. A total sample of 1,026 commercial vehicles in a large variety of types, weights, and horsepowers was investigated.

A study of weight-power ratios published in 1957(*1*)[2] included data collected in 1949, 1950, and 1955. The 1950 data, collected in conjunction with the annual truck weight survey, contained information on 10,726 trucks in 39 States. The 1949 and 1955 data, were obtained from brake tests on 782 and 862 commercial vehicles, respectively, in the same States as the 1963 brake study. The 1963 brake test data have been used to update the weight-power ratio information published in 1957 and to indicate the trend in the ratios.

It is believed that a performance requirement of 400 pounds per horsepower for commercial vehicle operation would improve the weight-power ratios, but a substantial reduction in this ratio will be required before the two types of vehicles attain similar performance levels. For example, large horsepowers are required for trucks to maintain a high speed on grades. A trailer combination having a gross weight of 100,000 pounds and powered by a 250-net-horsepower engine can maintain a speed of 50 m.p.h. on the level, but up a 3-percent grade only 20 m.p.h. To maintain the 50 m.p.h. speed up the 3-percent grade, this vehicle would require an engine capable of producing a net horsepower of 700.

Although production of highway commercial vehicles equipped with 700-horsepower engines may be remote, the trend is toward larger engines and smaller weight-horsepower ratios. Furthermore, the authors believe industry is capable of producing engines larger than those now in use. In addition, information has been developed to show that an increase in the average road speeds is economically justifiable for owners who can use their equipment advantageously during the time saved (HRB Bulletin 301, *Line-Haul Trucking Costs in Relation to Vehicle Gross Weights*)

Conclusions

On the basis of information gathered in the study reported here, the authors conclude that commercial vehicles having larger horsepower engines soon may be operating on the highways. Use of these larger engines would narrow the gap between the performance of passenger cars and commercial vehicles. Criteria developed during earlier studies on vehicle performance and still accepted in designing highways are: (1) operating characteristics of commercial vehicles and passenger cars are not similar; (2) these two types of vehicles cannot be designed any time soon to similar performance standards without an injustice being done to one or both; (3) public interest requires that highways be adequately designed and constructed to serve both passenger vehicles and commercial vehicles; (4) by appropriate highway design, the highways required for operation of both types of vehicles can be designed so that these vehicles can operate without undue movement restrictions.

Analysis of the data collected during the study on the weight-horsepower ratios has provided information from which the authors concluded that the performance gap is being narrowed. The weight-power ratios of commercial vehicles decreased 12 percent from 1949 to 1955 and 28 percent from 1955 to 1963.

Purpose of Study

The primary purpose of the study was to update information on weight-power ratios of commercial vehicles in order to analyze traveltime, grade-climbing ability, and accelerating ability of trucks. Another purpose was to investigate the trend in weight-power ratios, based on 1949 and 1955 brake tests and 1950 truck weight survey information. Data from the 1963 study may also be used to determine the effect of minimum weight-power ratio requirements on the trucking industry. It can provide information on the percentage of vehicles affected by a minimum performance requirement. For example, a performance requirement of 400 pounds gross weight per net horsepower can be translated into grade-climbing ability because it is inversely proportional to the ratio. A weight-power ratio of 400 is approximately equal to 20 m.p.h. on a 3-percent grade.

Many believe that the result of a minimum performance requirement would be better balance between the vehicle weight and the load for which its tires, brakes, and other components were designed. It is doubtful that commercial vehicles could ever be required to maintain the same speed on grades as passenger cars. Nevertheless a minimum performance requirement may provide the highway engineer with a level of vehicle performance on which to base highway design standards conducive to safer and more efficient movement of traffic.

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[1] [illegible] October 1964 meeting of National Transportation [illegible] and Fuels and Lubricants Meeting, [illegible] of Engineers, Inc., Baltimore, Md.

[2] [illegible] Weights of Motor Trucks and Their Horsepower [illegible] Public Roads, A Journal of Highway Research, vol. 29, No. 10, October 1957, pp.233–238

Analysis Procedure

The first step in the analysis of the data collected was to determine the net horsepower of each truck in the sample. Whenever possible, this was obtained in the field from the vehicle manufacturers rating plate. The net horsepower for unrated vehicles was determined from the vehicle specifications of individual manufacturers and the Automobile Manufacturers Association. When only gross horsepower could be determined, the net horsepower was assumed to be 90 percent of that value. When two or more horsepower options were available for a given model and it was not possible to determine which was installed in the particular vehicle, the net horsepower of the smaller engine was used for computing the ratio.

After computation of weight-power ratios, the ratios were grouped according to vehicle type and gross weight. The average net horsepower, the average gross weight, and the average weight-power ratio for each vehicle type were computed. The gross weights, net horsepowers, and weight-power ratios for each vehicle type were tabulated also. Cumulative frequency distributions of weight-power ratios were made for each vehicle type by grouping the weight-power ratios in class intervals of 50 pounds gross weight per net horsepower. The 15th, 50th, and 85th percentiles of the frequency distributions for 1955 and 1963 were tabulated and compared.

An analysis was made to determine the relation between the weight-power ratio and the gross weight of the vehicles regardless of vehicle type. The vehicles were grouped in intervals of 10,000 pounds gross weight and the average ratio was calculated for each interval group. These weight-power ratios were plotted in relation to the gross weight and compared with similar curves derived from 1949 and 1955 data. A separate analysis was made for only loaded vehicles in the 1963 braking study. Loaded vehicles were those that carried any cargo or payload. Of the 1,026 vehicles, 634 were loaded. The same procedure for analysis of loaded vehicles was used as for the total sample of empty and loaded vehicles.

Survey Results

A summary of the horsepowers, weights, and weight-power ratios for each vehicle type is shown in figure 1 and table 1. Two methods of listing the data were used in figure 1: average for all vehicles and average for loaded vehicles only. Gross weights, net horsepowers, and weight-power ratios increased as the number of axles increased

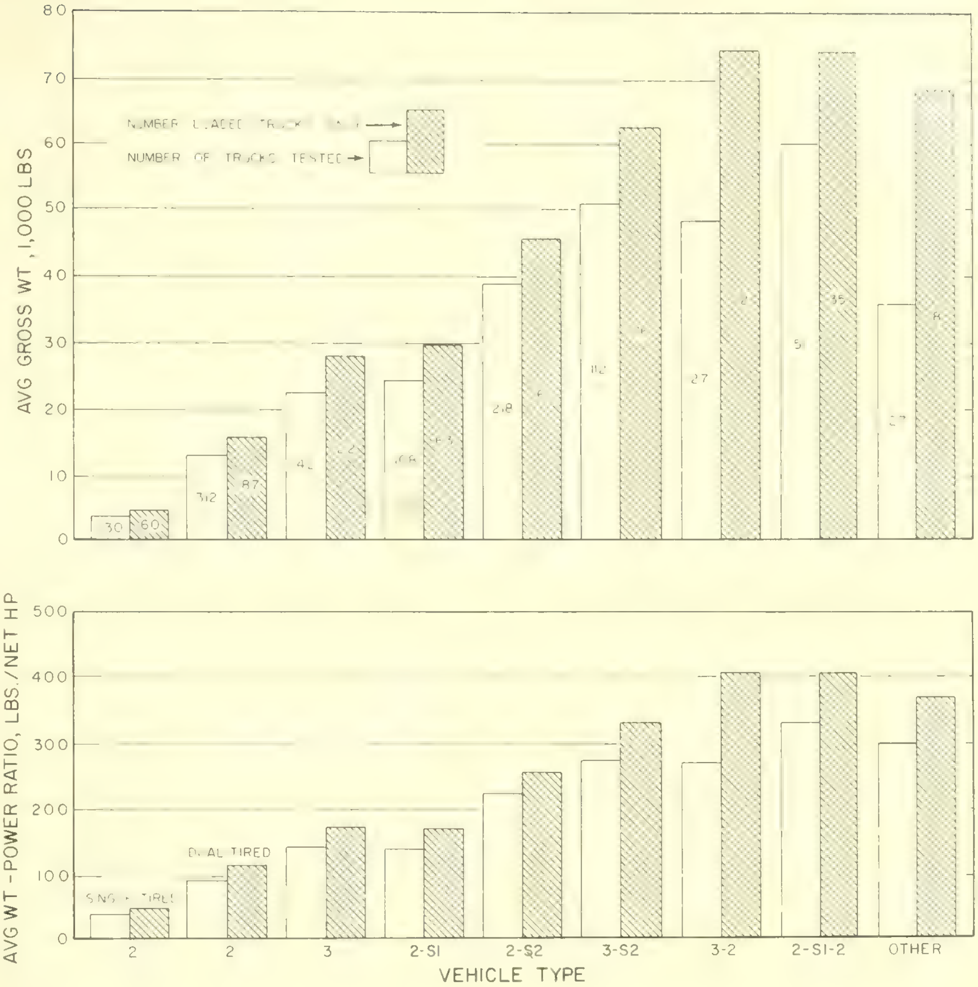

Figure 1.—Average weight-power ratios and gross weights for commercial vehicles, 1963 brake test.

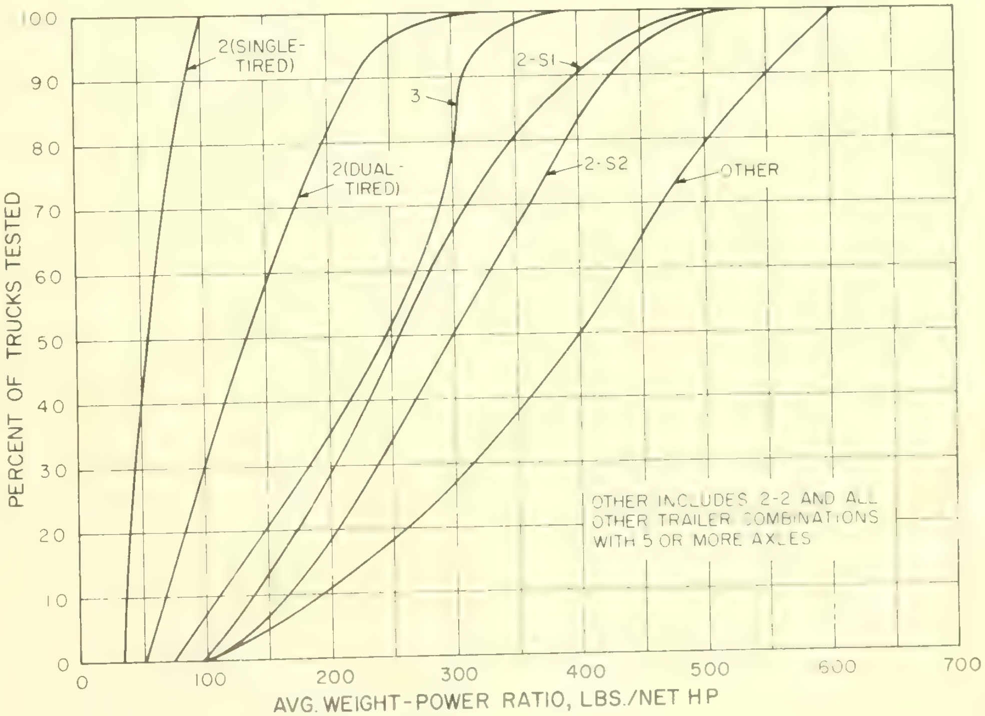

Figure 2.—Cumulative frequency distributions of weight-power ratios for all commercial vehicles, 1955 brake test.

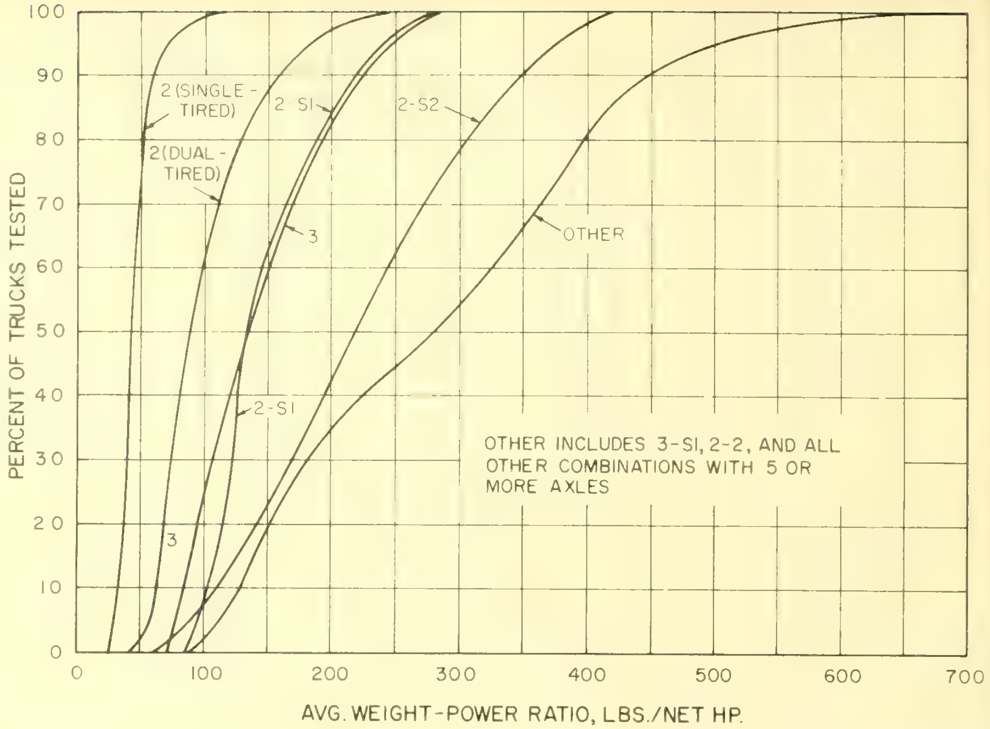

Figure 3.—Cumulative frequency distributions of weight-power ratios for all commercial vehicles, 1963 brake test.

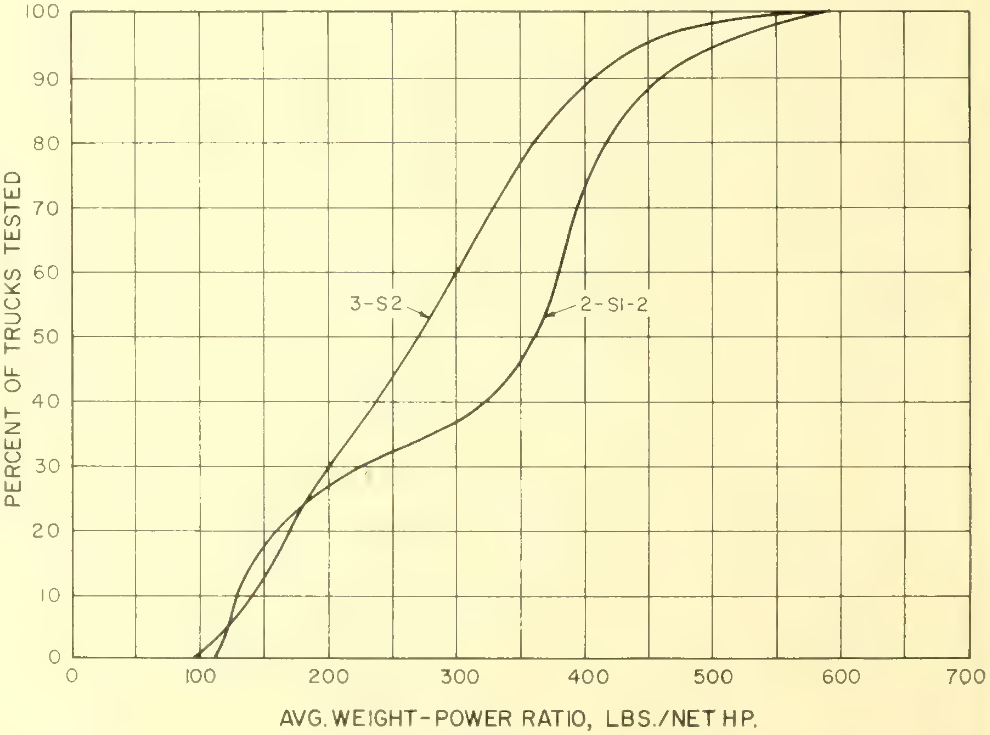

Figure 4.—Cumulative frequency distributions of weight-power ratios for all 3-S2 and 2-S1-2 trailer combinations, 1963 brake test.

up to 5 axles, as shown in table 1. For vehicles having 5 or more axles, the measures remained fairly constant. Thus, a large variation in hill climbing and accelerating ability was indicated for the different vehicle types. The smaller weight-power ratios were generally computed for empty vehicles having large engines. For example, an empty 3–S2 trailer combination having a 310 net horsepower engine weighed 29,100 pounds and had the smallest weight-power ratio, 94, for any of the 3–S2 trailer combinations tested. The 310 net horsepower also was the largest horsepower for any vehicles tested in the 1963 braking performance study.

Data for the loaded vehicles only are shown in table 1. Although the average weight and the lowest limits of the weight range were larger for the loaded vehicles than for the total of all vehicles, net horsepower remained nearly constant for both loaded only and the total of all vehicles tested. Therefore, the average weight-power ratio and the lowest weight-power ratios were also larger for the loaded vehicle sample than for the total vehicle sample.

Larger weight-power ratios occurred when vehicles having small engines were heavily loaded. A 3–S2 trailer combination having a gross weight of 94,650 pounds and a net engine horsepower of 135 had a weight-power ratio of 701, the largest ratio for any vehicle in the 1963 brake study. This particular vehicle was operating under a special permit because it had tandem axle weights in excess of the legal limit for the State in which it was operating. However, it is possible to transport extremely heavy loads and remain within the legal weight limits of the State, as illustrated by a 3–S3–5 trailer combination having a gross weight of 132,570 pounds and a weight-power ratio of 567.

Cumulative frequency distributions for all vehicle types for 1955 and 1963 are shown in figures 2 and 3, respectively. The summary in table 2 of the cumulative frequency distributions at the 15th, 50th, and 85th percentiles shows a reduction in the ratios from the 1955 to the 1963 study for all vehicle types. The percentage change at the 50th percentile was 28 percent for 2-axle, single-tired trucks; 36 percent for 2-axle, dual-tired trucks; 45 percent for 3-axle trucks; 48 percent for 2–S1 trailer combinations; 27 percent for 2–S2 trailer combinations; and 31 percent for all other trailer combinations. The average reduction in ratios for all vehicle types was approximately 30 percent at the 15th and 50th percentiles and 25 percent at the 85th percentile.

Cumulative frequency distributions of weight-power ratios for trailer combinations 3–S2 and 2–S1–2 from the 1963 study are shown in figure 4. These curves show data separated from that shown by the curve labeled OTHER in figure 2. The sample size for the 1955 study was not large enough for such a breakdown. The irregularity in the curve for the 2–S1–2 trailer combinations (fig. 4) occurred because nearly all of these trailer combinations either were traveling empty or heavily loaded. The sample vehicles in the middle-weight range was small; only 5 of 51 had gross weights within the range of 35,000 to 70,000 pounds.

Cumulative frequency distributions of weight-power ratios for only loaded vehicles in the 1963 study are shown in figure 5 by vehicle type. The curve designated as OTHER includes 3–S1, 3–S2, 2–2, 2–S1–2, and all other trailer

Table 1.—Range and average of gross weights, net horsepowers, and weight-power ratios for commercial vehicles weighed in 1963 brake performance study

Commercial vehicles	All commercial vehicles tested in 1963 brake study						Loaded commercial vehicles tested in 1963 brake study					
	Gross weight		Net horsepower		Weight-power ratio		Gross weight		Net horsepower		Weight-power ratio	
	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average
	Pounds	*Pounds*	*Horsepower*	*Horsepower*	*Ratio*	*Ratio*	*Pounds*	*Pounds*	*Horsepower*	*Horsepower*	*Ratio*	*Ratio*
2 (single-tired)	2, 545–11, 120	4, 795	50–165	109	24-128	44	3, 270–11, 120	5, 275	63-165	108	29–128	49
2 (dual-tired)	5, 700–31, 410	13, 230	80–198	136	42-267	97	6, 020–31, 410	15, 425	80–198	136	45–267	113
3	11, 000-47, 410	22, 785	95-222	157	71, 282	145	11, 000–47, 410	27, 460	95–222	157	82-282	175
2–S1	13, 000–45, 400	24, 630	118–230	165	84-304	149	14, 500–45, 400	28, 700	118–230	167	99–304	172
2–S2	15, 900–64, 805	39, 030	110–238	172	89–427	227	19, 270–64, 805	44, 625	110-235	172	120–427	259
3–S2	22, 010–94, 650	50, 625	128–310	184	94–701	275	27, 240–94, 650	60, 775	134-255	185	151–701	329
3–2	22, 400–78, 200	48, 070	128–250	184	93-511	261	49, 600–78, 200	73, 150	150–209	182	329-511	403
2–S1–2	24, 500–82, 770	59, 595	130–235	186	111–590	321	36, 600–82, 770	73, 685	134-235	185	203-590	398
Other trailer combinations [1]	16, 000–132, 570	54, 995	153-288	188	88–625	292	16, 000–132, 570	67, 285	133–234	187	88–625	359

[1] Includes 3–S2 and 2–S1–2 and other trailer combinations not listed specifically.

Table 2.—Comparison of weight-power ratios by percentiles from cumulative frequency distributions for 1955 and 1963

Commercial vehicles	Weight-power ratio, pounds per horsepower					
	15th percentile		50th percentile		85th percentile	
	1955	1963	1955	1963	1955	1963
2 (single-tired)	41	32	58	42	85	56
2 (dual-tired)	73	64	135	87	208	142
3	132	88	245	135	306	208
2–S1	161	108	256	133	376	204
2–S2	186	126	300	218	406	327
3–S2	.	157	.	272	.	377
2–S1–2	.	141	.	360	..	431
Other trailer combinations [1]	232	138	400	278	531	428

[1] Includes 3–S2 and 2–S1–2 and other trailer combinations not listed specifically.

Table 3.—Weight-power ratios for loaded vehicles from cumulative frequency distributions, by percentiles, 1963

Commercial vehicles	Weight-power ratio, pounds per horsepower		
	15 percentile	50 percentile	85 percentile
2 (single-tired)	37	45	60
2 (dual-tired)	75	106	157
3	110	125	243
2–S1	117	116	223
2–S2	180	252	346
3–S2	247	315	408
2–S1–2	338	388	454
Other trailer combinations [1]	251	354	452

[1] Includes 3–S2 and 2–S1–2 and other trailer combinations not listed specifically.

combinations having 5 or more axles. The 3–S2 and 2–S1–2 types are shown separately because they are the largest groups for which data is contained in the curve labeled OTHER. The 15th, 50th, and 85th percentiles of the cumulative frequency distributions for the loaded vehicles tested in 1963 are summarized in table 3. These ratios follow a pattern similar to that for the total of vehicles tested in 1963, except that the ratios are larger when only loaded vehicles are considered.

In figure 6 the trend in weight-power ratios from 1949 to 1963 is illustrated. The curves are based on average data for all commercial vehicles weighed in the brake studies of 1949, 1955, and 1963. The average ratios for all vehicles sampled in the 1950 truck weight survey are indicated by the triangular symbols in figure 6. Average ratios for the 1950 truck weight survey closely follow the curve for the 1949 brake test data; this indicates the validity of the smaller sample of vehicles. The average ratios for all vehicles sampled in a different, but related, study conducted in 1964 near Woodbridge, Va., also are shown in figure 6. These data on 408 trucks are indicated by the circular symbols. Data collected at Woodbridge closely approximate the 1963 brake test data and, therefore, substantiate the results of the 1963 brake test.

The reduction in the weight-horsepower ratios from 1949 to 1955 amounted to about 15 percent for vehicles having gross weights less than 40,000 pounds. Above that weight, the change decreased to about 8 percent at 80,000 pounds. From 1955 to 1963, the reduction amounted to about 25 percent for gross weights up to 40,000 pounds. The change gradually decreased to about 16 percent at 80,000 pounds gross weight.

The 1963 test data on loaded trucks also were analyzed. The curve obtained closely approximated the curve for all trucks in the sample. At weights of less than 40,000 pounds, the difference in the two curves was 2 percent or less. For weights of more than 40,000 pounds, the two curves are identical. Therefore, only one curve is shown.

The trend in average weight-power ratios by vehicle type from 1949 to 1963 is shown in table 4. The variation in percentage change from 1949 to 1955 and from 1950 to 1955 was small, except when the sample size was small. The comparison of 1949 and 1955 data showed that the largest reductions in ratio occurred for 2-axle, single-tired trucks and 2–S2 and

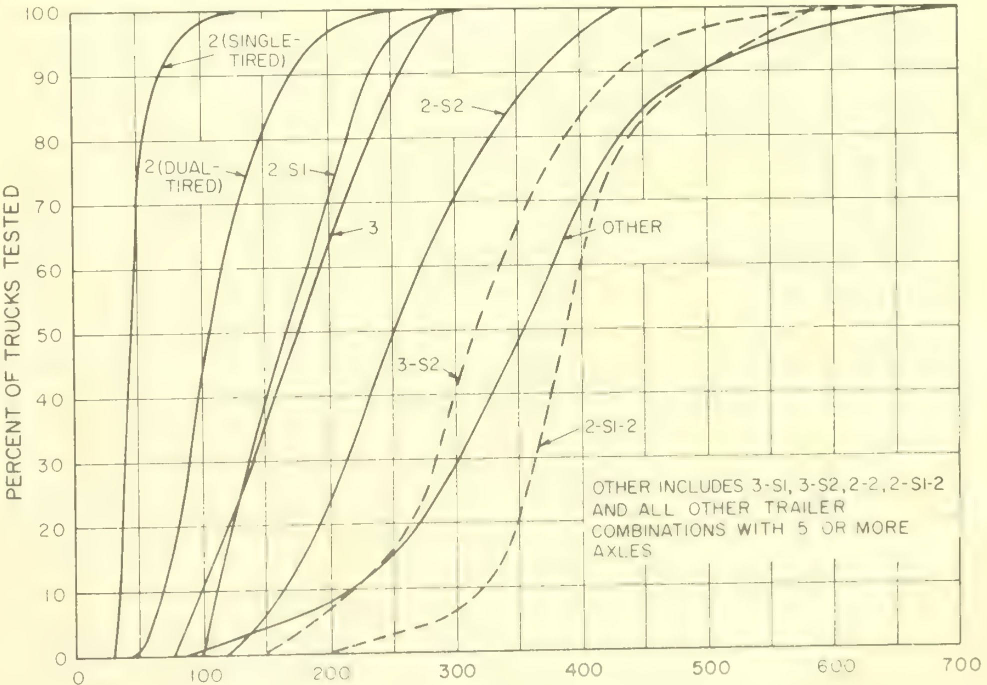

Figure 5.—Cumulative frequency distributions of weight-power ratios for loaded commercial vehicles, 1963 brake test.

3–S2 trailer combinations. The ratio increased from 1949 to 1955 for 3-axle trucks and 2–3, 3–2, and 2–S1–2 trailer combinations. The ratio for 2-axle, dual-tired trucks, did not change. A reduction in the ratio occurred for all vehicle types from 1955 to 1963, the largest percentage reductions were for the 2-axle, dual-tired, and 3-axle trucks, and 2–S1 trailer combinations. The overall reduction in the ratio from 1949 to 1955 was about 12 percent. The corresponding reduction from 1955 to 1963 was approximately 28 percent.

The percentages of vehicles sampled in 1955 and 1963 that could not meet performance requirements are listed in table 5. Comparison of these percentages shows considerable change. Percentages for 3–S2 and 2–S1–2 trailer combinations are not shown for 1955 because of inadequate samples. In 1955, 50 percent of the vehicles having 5 or more axles—14 percent of the total sample—had weight-power ratios of more than 400 pounds per horsepower. In 1963, only 20 percent of vehicles with 5 or more axles, and 5 percent of the total sample, had weight-power ratios of more than 400. The percentage of the loaded vehicles sampled in 1963 that could not meet the different performance requirements are shown in table 6. These percentages were taken from figure 5. In 1963, 30 percent of the loaded vehicles with 5 or more axles and 8 percent of the total sample of loaded vehicles had weight-power ratios of more than 400.

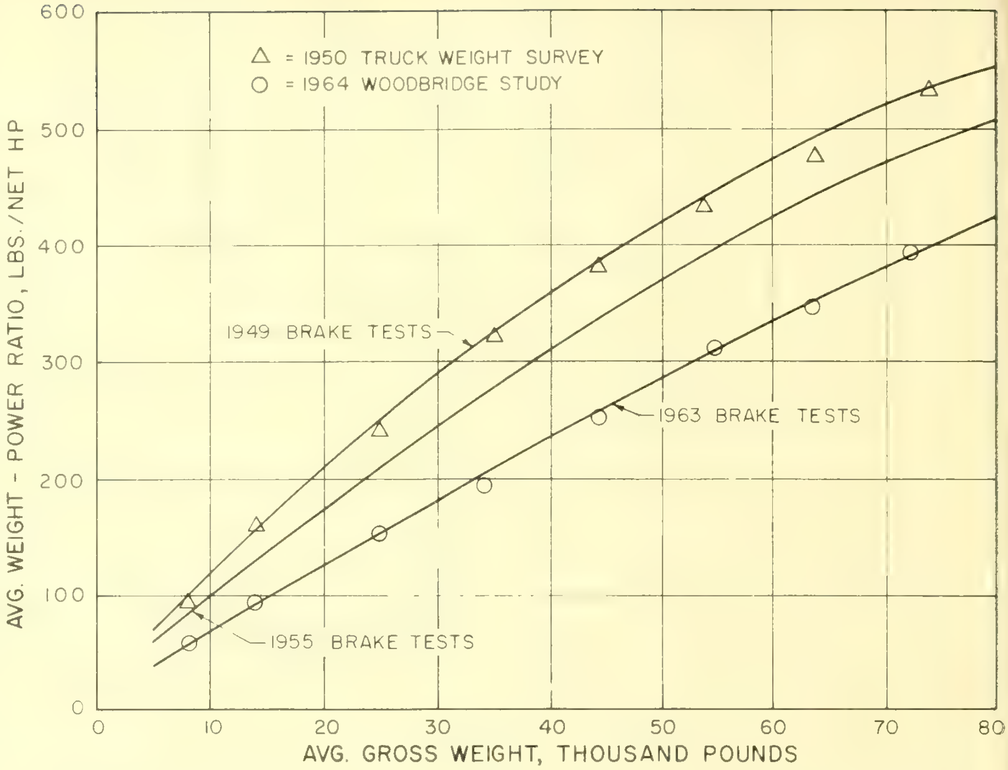

Figure 6.—Trend in weight-power ratios from 1949 to 1963 based on average data for all types of commercial vehicles.

Table 4.—Average weight-power ratios for all vehicles by types for 1949, 1950, 1955, and 1963

Commercial vehicles	Number of vehicles				Average weight-power ratios				Percentage reduction of weight-power ratios		
	1949	1950	1955	1963	1949	1950	1955	1963	1949–55	1950–55	1955–63
2 (single-tired)	19	239	99	130	81	75	57	44	30	24	23
2 (dual-tired)	275	3,642	272	312	142	135	142	97	0	–5	32
3	38	263	67	42	227	244	231	145	–2	5	37
2–S1	228	3,900	117	108	291	294	264	149	9	10	44
2–S2	87	1,991	145	217	369	357	301	227	18	16	25
3–S2	46	483	57	112	422	411	348	275	18	15	21
2–3, 3–2, and 2–S1–2	51	136	71	78	394	384	418	300	–6	–9	28
Other trailer combinations[1]	38	72	34	27	428	421	374	292	13	11	22
TOTAL	782	10,726	862	1,026							
WEIGHTED AVERAGES					260	253	228	165	12	10	28

[1] Includes trailer combinations not listed specifically.

Table 5.—Percentage of all vehicles of given types weighed in the 1955 and 1963 brake tests that could not meet indicated performance levels

Commercial vehicles	Vehicles with weight-power ratios larger than—											
	250:1		300:1		350:1		400:1		450:1		500:1	
	1955	1963	1955	1963	1955	1963	1955	1963	1955	1963	1955	1963
2 (single-tired)												- -
2 (dual-tired)	3		1									- -
3	48	10	21		2		---					- -
2–S1	53	2	34	1	20		10		2	..		..
2–S2	66	37	50	21	34	10	17	3	5		1	- -
3–S2		57		38		23		12		5	..	6
2–S1–2		67	---	65		55		28		12	----	6
Other trailer combination[1]	82	55	73	46	62	34	50	20	35	10	22	7
TOTAL	38	20	29	14	20	9	14	5	8	2	4	1

[1] Includes 3–S2 and 2–S1–2 and other trailer combinations not listed specifically.

Table 6.—Percentage of loaded vehicles weighed in the 1963 brake tests that could not meet indicated performance levels

Commercial vehicles	Loaded vehicles with weight-power ratios larger than—					
	250:1	300:1	350:1	400:1	450:1	500:1
2 (single-tired)						
2 (dual-tired)	----			----		----
3	12					---
2–S1	4	1				
2–S2	51	29	14	4	----	
3–S2	84	59	34	17	7	3
2–S1–2	97	94	80	40	18	9
Other trailer combinations[1]	85	71	52	30	15	9
TOTAL	33	23	15	8	3	2

[1] Includes 3–S2 and 2–S1–2 and other trailer combinations not listed specifically.

Offtracking Calculations For Trailer Combinations

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

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In this article the offtracking characteristics of single-unit trucks and trailer combinations are described. Offtracking results were obtained by use of scale models of vehicles making turns on radii ranging from 25 to 255 feet. Individual vehicle offtrackings are influenced by three variables: the degree of turn, the length of vehicle wheelbase, and the turning radius. It was determined that the offtracking measurements of a trailer combination may be calculated by adding the offtracking measurements of the individual vehicles in the combination. Also, the offtracking is greatest when the projection of the rear axle axis passes through the turning radius center, even though the projections of the other axles on the vehicle or trailer combination do not pass through the turning radius center at the same time.

The research reported here was planned to develop a more simple, quick, and comprehensive method for calculating offtracking. This method uses a series of figures constructed to allow for direct reading and calculation of offtracking for almost all practical highway vehicles and trailer combinations. The information is given for turns of 90 degrees and 270 degrees and for outer front-wheel turning radii from 25 to 225 feet. The range of turns and turning radii covers most of the vehicle turns made on city streets and turns made at rural intersections, including at-grade intersections, diamond interchanges, and separated cloverleaf interchange ramps.

Introduction

EXPANSION of the National System of Interstate and Defense Highways and increasing use of the nation's highways has placed new demands on highway designers. Design emphasis is on highways that will permit good traffic flow, traveling ease and also provide maximum safety. Changes also are being made in the vehicles that use the highways. Size and weight regulations for commercial vehicles are being reevaluated and changed, and more and larger trailer combinations are using the highways. As a result more information is required on the handling characteristics of the larger vehicles.

The turning and offtracking characteristics of single-unit trucks and trailer combinations are of particular interest to the highway engineer as he must use this information in the design of highway curves, city street turns, and freeway entrance and exit ramps. Until recently only limited information on turns of different degrees and turning radii has been readily available. Data from the vehicle manufacturers have been sparse. Several studies have been made on vehicle steering performance but most of the material reported has provided information only on minimum radius turns on the operation of a few specific vehicles and trailer combinations. The data presented usually related only to resultant maximum offtracking without regard to the degrees of turn made by the vehicle before it exited onto a tangent.

Sketches, drawings, and detailed descriptions of the minimum turning paths of specific vehicles have also been provided. Although of value this information is inadequate because easy interpolation cannot be made of offtracking measurements between different types of vehicles and neither can comparisons be made of performance of vehicles of different wheelbases operated on turns of different radii. Even the SAE offtracking formulas (*1*) [2] require the use of specific vehicle dimensions, and then only maximum offtracking measurements are obtained for a particular combination. Thus, to make a comparison of the offtracking characteristics of different vehicles or to determine the offtracking limits for some particular turning radius, a long and tedious process of individually calculating the offtracking for each variation in vehicle dimensions is required.

Previously reported offtracking data have been based on measurements taken from the center of the axles of the vehicle. Although such data may be satisfactory for automotive engineering uses, the highway engineer must add or subtract other factors. In the study reported in this article all offtracking and turning radius measurements were made to the outside of the outer tire of an axle.

Conclusions

On the basis of findings from the research reported here, the authors have concluded that additional research may be required. Information on offtracking for turns of different degrees—only 90- and 270-degrees turns were studied—can be obtained by using similar models and procedures. They suggest that perhaps studies should be made of the maneuverings required for long trailer combinations on different types and sizes of cloverleaf intersections.

Although the research results reported here indicate that width over the tires has almost no effect on the offtracking characteristics of the outside of the outer tires of a trailer combination, additional studies may be required because of offtracking of certain units. It is believed that the width over the front tires of a power vehicle having Ackerman steering may have some limiting effect on the offtracking of the vehicle.

The authors also believe that additional research is needed to define more precisely the percentage relation between the turning radii and the wheelbase of different trailers. Knowledge gained from such a study would be useful to highway designers so that trailer backup and pivot motion on 180- and 270-degree turns could be prevented.

Basis of the Study

The fundamental premise made for the study was that: The sum of the offtracking of the individual vehicles of a highway trailer combination closely approximates the total offtracking of the combination. The research plan included experiments with vehicle models that led to the establishment of patterns of vehicle offtracking behavior that are related to differences in wheelbase length, turning radius, and degree of turn. The final step in the research was the development of methods of plotting these data for rapid use and comparison.

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[1] Presented at the 45th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1966.

[2] References indicated by italic numbers in parentheses are listed on page 100.

Because sometimes two or more engineering organizations have defined the same terms differently and a few terms are used that have not been previously defined, the definitions in the following list and those at the front of this magazine should be considered carefully.

Angle of turn.—The angle of turn is the angle through which a vehicle travels in making a turn (2).

Axle.—For simplification, only the single term axle is used: it designates either a single axle or the centerline between tandem axles; this application depends on the vehicles being considered. The term axle can be used because the theoretical turning center of a tandem-axle assembly lies on the centerline between tandem axles.

Cramp angle.—The cramp angle is the limit of the turning ability of the front wheels of an Ackerman-type front axle and is limited by the construction of mechanical parts around the the front-axle, kingpin-pivot mechanisms. These constructions limit the degree to which the inner front wheel may be turned and also the turning of the outer front wheel.

Fifth wheel.—The fifth wheel is a lubricated bearing plate, mounted on a tractor chassis or on a trailer converter dolly chassis, arranged with an internal clutch device to engage and hold the kingpin of a trailer. The fifth wheel clutch engages and locks upon contact with the trailer kingpin; a manual release by the driver is required to separate the trailer kingpin and the fifth wheel. Primarily, the manual fifth wheel is being used. Previously an automatic fifth wheel was attached to the trailer and connected to this landing gear, and the kingpin was mounted on the towing vehicle. Few of these automatic fifth wheels are in use now, and these few are used only in local cartage service where the semitrailers are a captive fleet. In the article on braking performance the term fifth wheel refers to a trailing, fifth-wheel, distance-measuring device.

Kingpin.—The term kingpin has two different meanings in automotive design, and the precise meaning is determined by the context in which the term is used. A kinpgin of a front axle of a power vehicle is a vertical or near vertical shaft. The shaft is the pivot connecting each stub axle that carries a front wheel of a power vehicle to the rigid center of an Ackerman-type front axle. All Ackerman-type front axles have two kingpins, one at each end of the rigid center of the front axle.

A kingpin of a trailer is a vertical pivot shaft attached near the front of and on the centerline of the underside of a trailer chassis. This kingpin is surrounded by a lubricated bearing plate. It engages a fifth wheel on a towing tractor, a trailer converter dolly, or it is permanently connected to the center of an undetachable front axle of a full trailer. A trailer is pulled by and pivots around its kingpin.

Radius of inside curb.—The radius of the inside curb is the radial difference between the turning radius and the turning track width when the offtracking of the vehicle is at the maximum amount for a given turn. This shortest inside curb radius will occur at only one point (instantaneously) on a 90-degree turn. On a 270-degree turn, the shortest curb radius may remain constant for some distance before the exit tangent.

[illegible] turning radius.—The minimum turning radius is th[illegible] of the minimum turning path of the outside of the outer front tire. Vehicle manufacturers data books usually give minimum turning radius to the centerline of the outer front tire.

[illegible] offtracking.—Negative offtracking occurs during a turn in which the radius of the path of the outer rear corner [illegible] becomes longer than the radius of the turning [illegible] outside of the vehicle's outer rear tire. For [illegible] a truck or a trailer that has a cargo body [illegible] the rear axle, negative offtracking occurs [illegible] rear of the cargo body swings outside the path of the outside of the outer rear tire.

[illegible] the path of the outside of the [illegible] that deviates toward the center of a turn from the circular path of the outside of the outer front tire, while the vehicle or trailer combination is making a turn.

Outside of tire.—The outside of a tire is the external side of a tire farthest away from the vehicle chassis.

Outside of outer tire.—The outside of the outer tire of a vehicle is the outside of the outermost tire on an axle on the outer side of a turn.

Outside of innermost rear tire.—The outside of the innermost rear tire of a vehicle is the outside of the rear tire nearest the turning radius center.

Overall length.—The overall length of a vehicle or trailer combination is the distance between the front bumper of the power vehicle and the rear bumper or guard on the rear vehicle.

Pintle hook.—A pintle hook is a vertical hook device attached to the rear of a tractive truck or to the rear of a leading (towing) semitrailer in a double trailer combination. The pintle hook engages the towing eye (ring eyelet) at the front end of the towbar of a trailer converter dolly or the towbar of a full trailer undetachable front-axle assembly.

Power vehicles.—Three general types of power vehicles are used:

- Single-unit trucks are power vehicles having Ackerman front-axle steering equipped with a cargo body but not equipped to pull a trailer.

- Tractive trucks are power vehicles with Ackerman front-axle steering equipped with a cargo body and a pintle hook that is attached to and recessed into the rear frame members so that a full trailer may be pulled.

- Tractors for commercial freight use are legally defined as truck-tractors to differentiate them from farm or industrial tractors. The single term tractor, however, is used in this article for a power vehicle of short wheelbase that is equipped with Ackerman front-wheel steering and a fifth wheel to engage and pull a semitrailer.

Rear axles of trailers.—Rear axles of trailers are attached primarily through springing suspensions and mechanisms to the trailer chassis so as to be in a fixed alinement with the longitudinal centerline of the trailer.

Rear overhang.—The rear overhang of a tractive truck, of a semitrailer, or of a full trailer is the distance between the centerline of the vehicle's rear axle and the centerline of its pintle hook.

Steering system.—One of two types of steering systems generally is used but related types are also used. The main ones are described in the following paragraphs.

- The Ackerman-steering system for front axles of power vehicles consists of a three-piece articulated axle with two front wheels that are mounted on short stub axles. The stub axles are attached to opposite ends of the rigid center section of the front axle by the front axle kingpins. The short stub axles are pivoted about the axle kingpins by steering arms and mechanisms connected to the driver's steering wheel.

- Fifth wheel pivot steering is similar to that used at the front ends of 2-axle, horse-drawn wagons. The front axle is a one-piece, rigid axle with the front wheels at each end of the axle. The rigid axle pivots about a kingpin located above the lateral center of the axle. Surrounding the kingpin are two lubricated bearing surfaces, the lower one is attached to the axle assembly. The upper bearing plate is attached to the underside of the vehicle chassis on its longitudinal centerline. These bearing plates give lateral and longitudinal stability to the cargo vehicle and make it possible for the trailer to be pulled by the kingpin. This type of steering is predominantly used at the front end of trailers.

- Pintle hook steering through a towbar is similar in action to fifth wheel pivot steering except that no vehicle weight rests on the pintle hook.

Trailers.—There are three types of trailers:

- A semitrailer is a cargo trailer equipped with one or more axles at or near its rear; it is constructed so that a substantial part of its tare weight and its cargo weight rests upon a tractor through the tractor fifth wheel.

- A full trailer is basically a semitrailer that has been converted into a full trailer by one of two methods. In one method the front axle and spring suspension are permanently connected to the chassis of the trailer. In the other method a semitrailer is combined with a trailer converter dolly.

- A trailer converter dolly is a very short wheelbase semitrailer. It consists of an axle attached through a spring suspension system to a platform (chassis) that carries a lower fifth-wheel plate. It has a towbar mechanism affixed at 90 degrees to its axle. The front end of the towbar is equipped with a towing eye that engages with a pintle hook on the rear of the towing vehicle.

Towbar.—A towbar is a bar, or a V-shaped assembly of two bars, attached to the chassis of a trailer converter dolly or to the undetachable front axle assembly of a full trailer, and constructed so that it has a towing eye at its forward end and exerts a pulling force in the middle of and at 90 degrees to the axle of a trailer converter dolly or to a full trailer undetachable front axle.

Turning path.—Turning path is the path of a designated point on a vehicle making a turn (*2*).

Turning radius.—The turning radius is the radius of the circular turning path of the outside of the outer front tire from the turning radius center.

Turning radius center.—The turning radius center is the point that is the center of the circular turning path followed by the outside of the outer front tire of the power vehicle.

Turning track width.—The radial distance between the turning paths of the outside of the outer front tire and the outside of the rear tire nearest the center of the turn is the turning track width (*2*).

Wheelbase.—The several measures of wheelbase, which depend on the type of vehicle, are defined in the following paragraphs.

- The wheelbase of a single-unit power vehicle (truck or tractor) is the distance between the centerline of the front axle and the centerline of the rear axle. The centerline between any tandem axles always is used as the reference point for wheelbase measurements.

- On semitrailers, the wheelbase is the distance between the trailer kingpin and the centerline of the rear axle.

- On trailer converter dollies, which are in effect short semitrailers, the wheelbase is the distance between the center of the towing eye of the towbar and the centerline of the dolly's axle.

- The wheelbase of full trailers is measured the same way as for semitrailers and is the distance between the kingpin of the trailer and the centerline of the rear axle.

- On complete trailer combinations, the overall wheelbase is the distance between the front axle of the power vehicle and the rearmost axle when the trailer combination is strung out in a straight line. This overall wheelbase may differ from the sum of the defined wheelbases.

Width over tires.—The width over tires is the outside-to-outside distance over the tires on an axle.

SAE definitions.—The meanings of the two following SAE terms are different from the definitions and measurements of offtracking that are used in this article. To prevent confusion the SAE definitions are given in the following statements.

Turning center, SAE.—The turning center is the point about which all parts of a vehicle or combination of vehicles revolve in describing a turn of constant radius and to which all wheel spindles are normally radial. For 2-axled bogies or tandems in which the axles are constrained to parallelism, the interaxle trunnion or its equivalent is assumed to be radial from this point (*3*). The location of this turning center moves around as a trailer combination enters a curve from a tangent, proceeds around the curve, and leaves on an exit tangent. This turning center should not be confused with the outer front wheel turning radius center referred to in this article.

Offtracking, SAE.—Offtracking is the difference in radii from the turning center to the vehicle centerlines at the foremost and rearmost axles of a vehicle or combination and represents the increase beyond the tangent track caused by a turn (*3*).

Peak offtracking.—Peak offtracking is the offtracking result obtained from the data shown in figure 5 and 6. This peak offtracking may be equal to or less than the maximum offtracking calculated by use of SAE formulas.

Fundamentals of Offtracking

Offtracking is the phenomenon in which the paths of the wheels of a rear axle of a single-unit power vehicle or of a trailer combination deviate inward toward the center of a turn from the circular turning path of the outside front wheel. When operating on turns uniform in radius, individual vehicles, whether in combinations or single-unit, offtrack in similar patterns of turns. The front wheels of a power unit do not offtrack, but all other axles on the vehicle or trailer combinations do. Although, most highway vehicles have nonsteerable rear axles, a

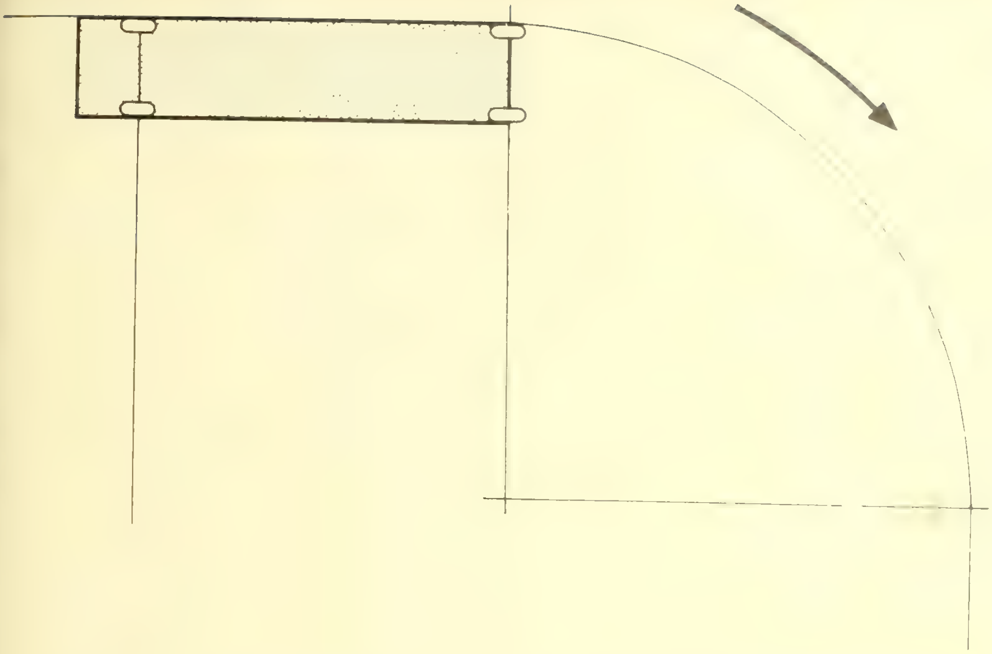

Figure 1.—A long wheelbase vehicle in tangent position about to enter turn.

small minority does have different methods of rear steering. In the study discussed in this article only vehicles with nonsteering rear axles were studied.

For practical vehicle-highway geometrics, the most important factor of offtracking is the offtracking that occurs when a single-unit vehicle or a trailer combination makes a turn of 270 degrees. On short wheelbase, single-unit vehicles, the peak offtracking may occur early in the first 90-degree segment of a turn; but on very long trailer combinations, the full 270 degrees of turn may be used before the peak offtracking occurs. For the longer trailer combinations, the offtracking during a 90-degree turn will be substantially less than their offtracking on a 270-degree turn. Also on 90-degree turns, the front wheels of the power vehicle of a long trailer combination will run for some distance on the exit tangent before the peak offtracking occurs. It is difficult to calculate the offtracking of trailer combinations having long wheelbases when the front wheels of the power vehicle travel on the exit tangent, but the solution can be obtained with scale models of vehicles. Of course, for very short wheelbase single-unit vehicles, which reach their maximum offtracking before 90-degrees of turn, any travel on the exit tangent does not increase the amount of offtracking.

On turns, the offtracking characteristics of single-unit vehicles and the individual vehicles in trailer combinations are affected by several interlocking factors, such as: (1) the degree of a turn; (2) the wheelbase of each individual vehicle in a trailer combination; (3) the uniform turning radius of the outside of the outer front tire of the power vehicle—this turning path of the outside of the outer front tire usually is the outer pavement or curb radius on a specific turn; (4) the radius of the outside of the outer front tire on a trailer's virtual front axle with reference to the turning radius center, when the towing vehicle is at its point of peak offtracking on a specific turn; (5) in trailer combinations, the rear trailing axle of each leading vehicle acts as a virtual front axle of the trailing vehicle.

The virtual front axles of trailing vehicles are: (1) on semitrailers, the tractor rear axle is the semitrailer's virtual front axle; (2) on trailer converter dollies or undetachable front axle assemblies of full trailers, the virtual front axle of such semitrailer-type assemblies is located on the centerline of the towing vehicle's pintle hook, which is the same location as the center of the pintle hook eye of the towbar; (3) on full trailers the axle of the trailer converter dollies is the virtual front axle. However, on undetachable front-axle assemblies, such a front axle of a full trailer is its real front axle. Both types of front axles for full trailers perform similarly.

Turning and Offtracking

Single-unit vehicles

The principles of offtracking for single-unit vehicles are illustrated in figures 1 through 4, which show the action on turns of vehicles with Ackerman-steering. A long single-unit vehicle is shown in figure 1 at its entrance tangent position just before entering a curve; the projections of the two stub axles of the front wheels and of the rear axle are parallel and do not intersect. For long vehicles, the projections of the front wheel, stub axles, and the rear axle vary from parallel when on the entrance tangent to different intersecting positions during a turn (fig. 2). The projections reverse toward parallelism when the

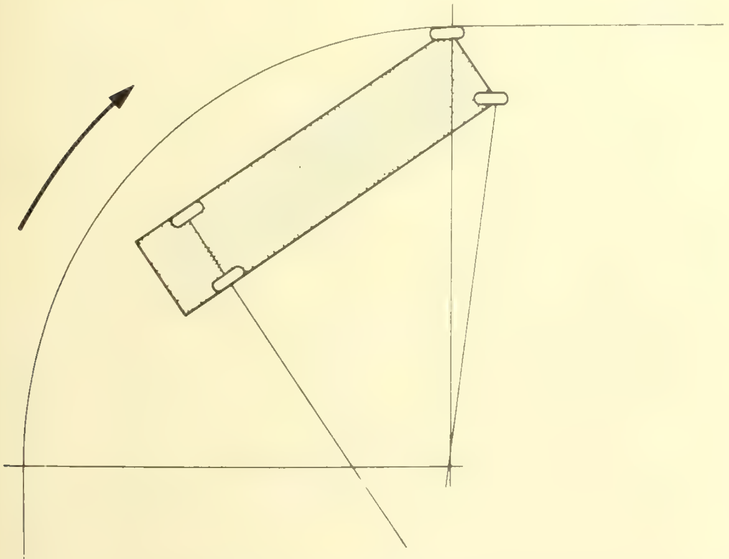

Figure 2.—A long wheelbase vehicle that has completed 90 degrees of turn but not reached peak offtracking.

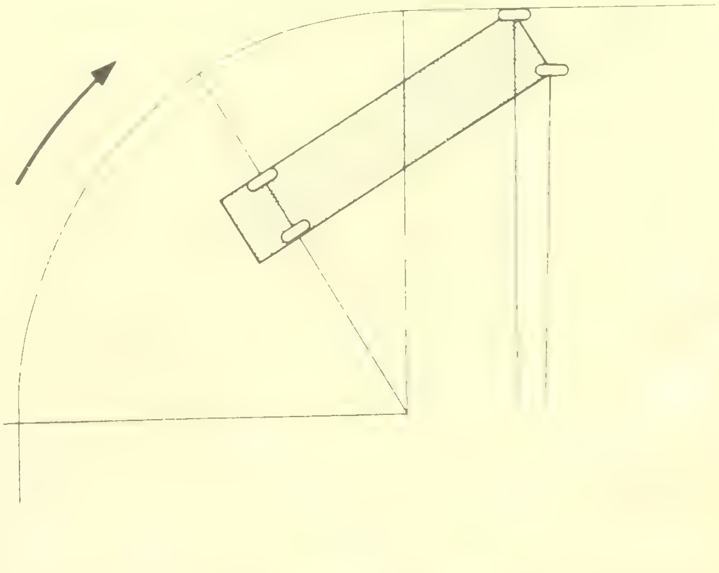

Figure 3.—A long wheelbase vehicle at its point of peak offtracking on an exit tangent.

front wheels leave the turn on an exit tangent (fig. 3). Thus, the offtracking rear wheels travel in a double spiral curve.

The vehicle shown in figure 2 has not attained its peak offtracking on a 90-degree turn; it is still in transition from its starting position even though the front wheels are at the exit tangent. Although, the projections of the stub axles of the front wheels pass through the turning radius center, the axis of the rear axle does not. In such situations, the peak offtracking will occur after the outer front tire of the vehicle is on its exit tangent (fig. 3). The front end of the vehicle has moved down the exit tangent until the projected axis of the rear axle passes through the turning radius center. This point of peak offtracking during a 90-degree turn was observed in the operation of the vehicle models. The axes of the front wheels no longer pass through the original turning radius center, but the axes of all axles will intersect at some distance behind the turning radius center. The amount of offtracking was measured with the vehicle models, but this offtracking cannot be calculated by use of the SAE equations.

For single-unit vehicles having short wheelbases, such as passenger cars and small trucks, maximum offtracking usually will occur during the first 90-degree segment of a turn (fig. 4). As shown, the axes of all axles intersect at the turning radius center. The offtracking of such vehicles was measured with the vehicle models and results are shown in figures 5 and 6. The offtracking of these short wheelbase vehicles also can be calculated by use of the SAE equations.

Trailer combinations

It is desirable that trailer combinations move continuously and progressively forward at a reasonably rapid speed when negotiating highway curves or at-grade intersections. Because of their jointed construction, trailer combinations may not travel in a continuous, smooth path when the turning radius is shorter than the trailer wheelbase. Such nonuniform type of travel is possible with trailer combinations because fifth-wheel, pivot-type steering permits a trailer to turn 90-degrees or more from the longitudinal axle of the towing vehicle. The angle through which the power vehicle can turn is limited by its steering cramp angle and its wheelbase.

An example of a trailer combination offtracking in a noncontinuous, irregular manner is illustrated in figure 7. As shown, when trailer combinations are negotiating 180-degree turns and the turning radius is less than the length of the trailer's wheelbase, the rear axle will pass behind the turning radius center and will pivot and travel backwards in an irregular path. The rear axle of long trailer combinations traveling on short radius, 270-degree turns also have similar backing and pivoting characteristics. Such reverse travel and pivoting of the rear axle can only be considered in very close quarters, for example, in buildings where the drivers carefully manipulate the trailer combinations at creep speeds. Data in figures 5 and 6 do not apply to this type of irregular offtracking.

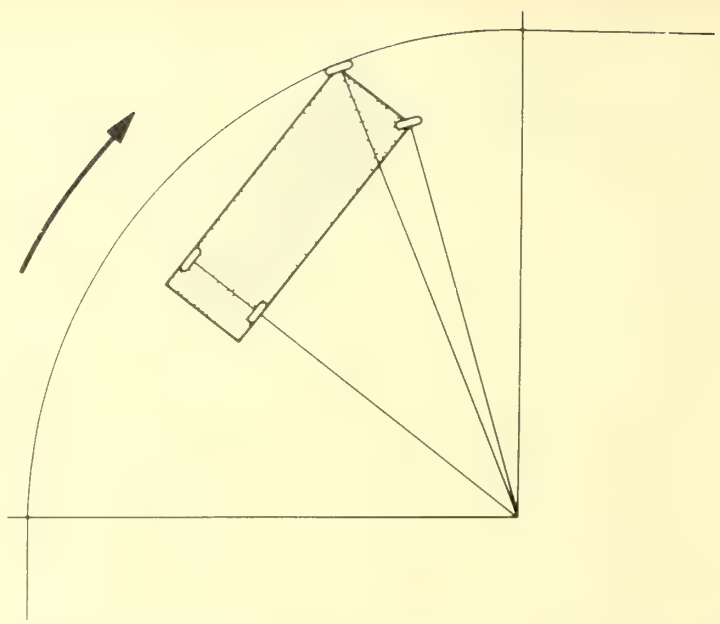

Figure 4.—A short wheelbase vehicle that has reached its point of maximum offtracking.

Trailer combinations negotiating 90-degree turns, however, travel in a continuous and smooth path regardless of which side of the turning radius center the semitrailer passes. A long wheelbase trailer combination following a relatively short turning radius, a situation truck drivers encounter as typical of city streets, is shown in figure 8. Because the outer rear tire on the rear axle passes behind the turning radius center, the maximum offtracking cannot be calculated with the SAE equations but can be and was measured with the vehicle models. The peak offtracking on such a turn occurs when the projection of the rear axle, as shown in figure 8, passes through the turning radius center although the front wheels of the power vehicle are on the exit tangent. The problems associated with long trailer combinations negotiating curves having short turning radii are troublesome, particularly on city streets and diamond approaches to controlled-access highways. Such problems will be magnified if, in the future, longer single-trailer combinations are permitted. In general, double-trailer combinations offtrack less than long single-trailer combinations.

Factors in Offtracking Determinations

An important feature of vehicle offtracking is that the peak offtracking for any degree of smooth and continuous turn occurs when a projection of the axis of the rear axle of a vehicle is on a radial passing through the turning radius center. This was observed with the vehicle models, which were equipped with a scale that projected from the outer end of the trailing rear axle. The peak outer rear tire offtracking occurred when the rear axle was parallel with a radial line passing through the turning radius center on the model test pattern.

The different measurements of offtracking data of interest and use to the highway design engineer are: (1) Dimensions of vehicles and trailer combinations; (2) turning radius of specified turn; (3) offtracking of trailing rear axle; (4) turning track width; and (5) inside curb radius, for zero clearance with tire.

Dimensions

The dimensions of vehicles and trailer combinations are needed so that the design engineer will know the sizes of vehicles to be considered in a specific turn situation. Dimensions needed of the individual vehicles in a trailer combination are: wheelbase of each vehicle, width over the tires; and on double cargo vehicle combinations, the rear overhang of each towing vehicle and the spacing between the vehicles.

The outer curb radius of a specific turn usually is determined by the location and terrain situation in the turning area. Offtracking is the radial distance between the outer front wheel turning radius of the outside of the outer front tire of a vehicle and the radius of the outside of the outer rear tire of a rear trailing axle, at the point of peak offtracking. Offtracking for single-unit vehicles and individual vehicles of trailer combinations can be obtained from the offtracking data in figures 5 and 6.

The turning width is the amount of offtracking plus the width over the tires of the dual tires on a rear axle or the width of the cargo body, if it is significantly wider than the width over the dual tires. This dimension was assumed to be 8.0 feet in the study

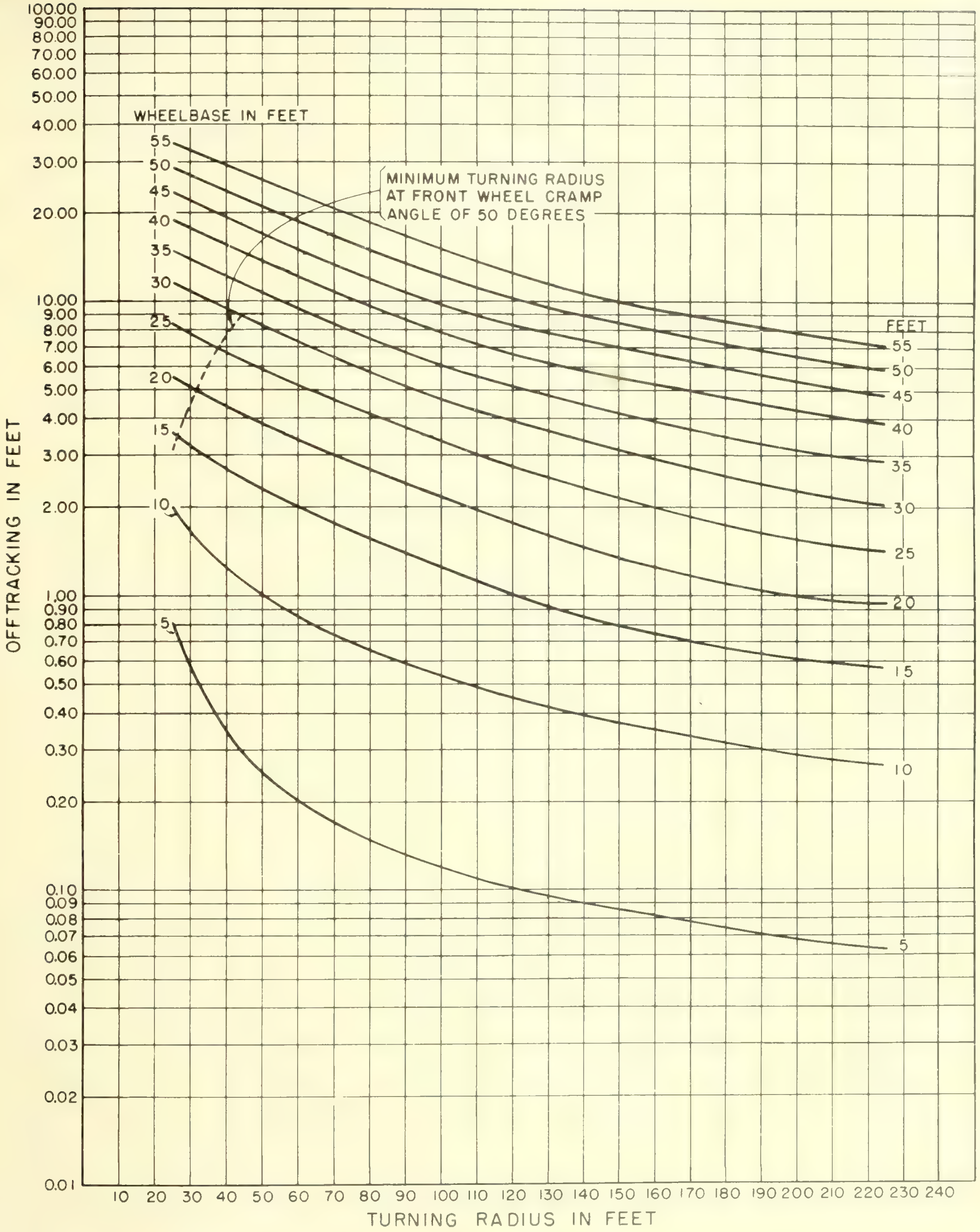

Figure 5.—Offtracking and turning radii for 90-degree turns and different wheelbases.

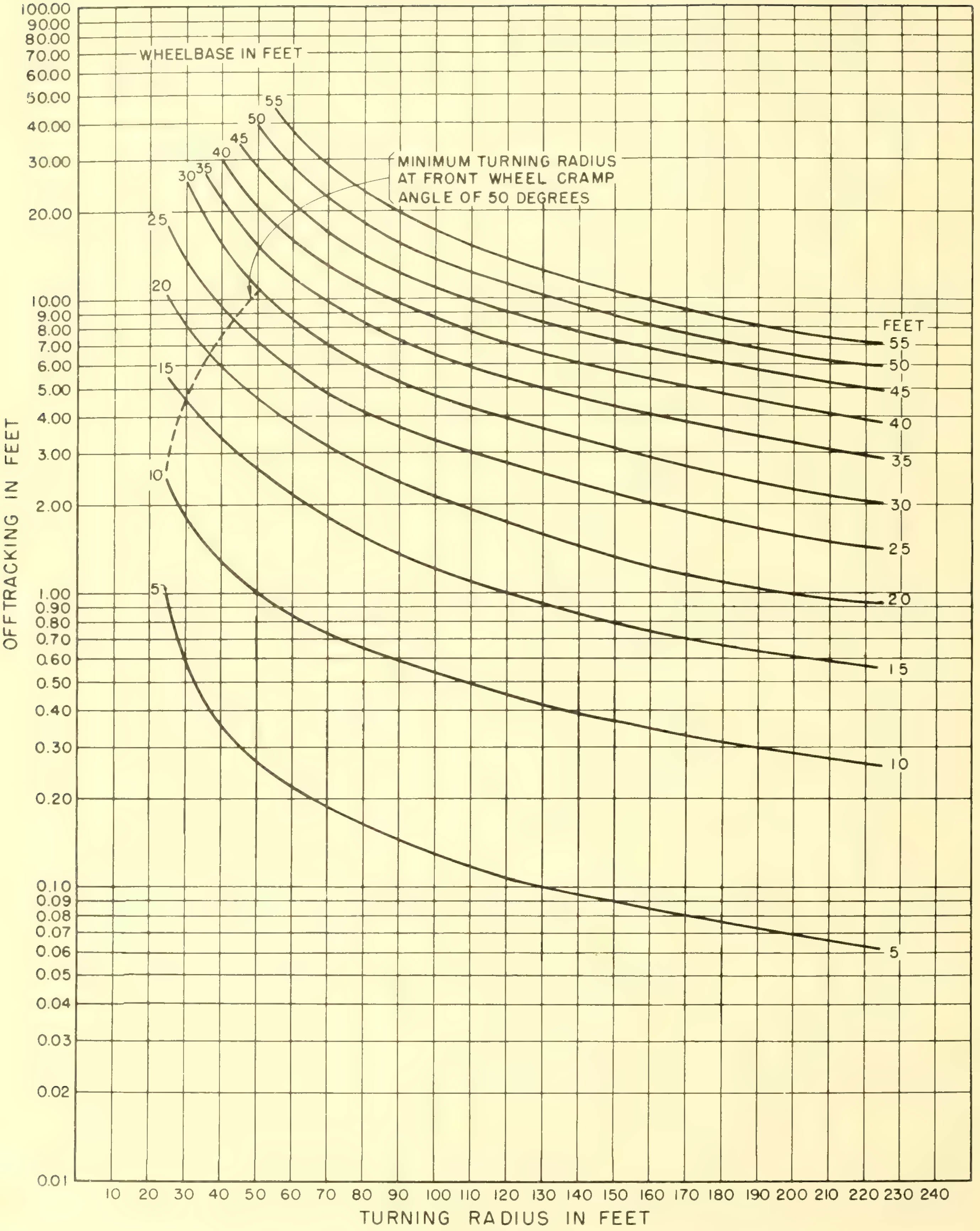

Figure 6.—Offtracking and turning radii for 270-degree turns and different wheelbases.

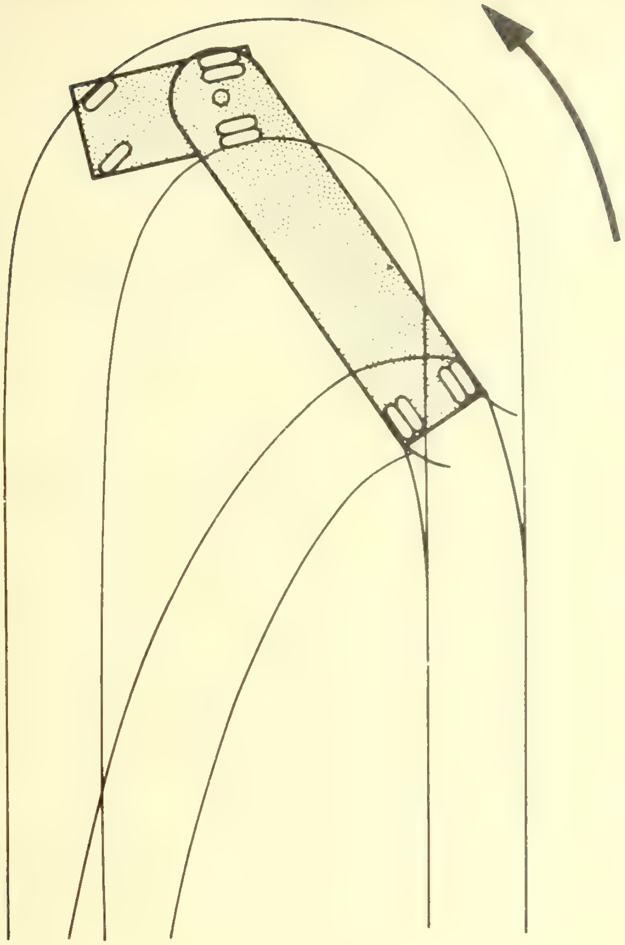

Figure 7.—A long wheelbase combination on a short radius turn, in which the semitrailer backs up and pivots behind the turning radius center.

reported here because it is the width presently most in use. However, the *1965 AASHO Size and Weight Recommendations* carry a provision for over-the-tire widths of 8.5 feet.

The inside pavement or curb radius on a turn is the radius from the turning radius center to the outside of the innermost rear tire on the rear axle at the point of its peak offtracking for a specific turn. The inside curb radius equals the original front-wheel turning radius minus the turning track width. This inside curb radius will permit a perfectly driven trailer combination, following the specified outer curb turning radius, to just clear the inner curb at its point of peak offtracking. The actual inner curb radius should be shorter so as to permit variations in driver manipulation. The offtracking of individual single-unit vehicles can be determined from the offtracking data in figures 5 and 6 by a single reference to either the 90-degree or the 270-degree information.

Trailer combination

The offtracking of a trailer combination on a specific turn is a summation of the offtracking of the individual vehicles in the trailer combination. Each vehicle in a trailer combination offtracks individually in accordance with its wheelbase and the radius from the turning radius center to the outside of the outer front tire on its virtual front axle. Determining the turning radius of the real or virtual outer front tire of each individual trailer vehicle in the train poses a problem. Because all trailing vehicles (semitrailers, trailer converter dollies, or undetachable full trailer, front-axle assemblies, and full trailers) offtrack and steer like semitrailers, a virtual or real front axle for each such semitrailer-like unit must be assumed. The point of peak offtracking for the rear axle of each towing vehicle on a specific turn will prescribe the turning radius of each following semitrailer-like unit. Proceeding from the front axle of a trailer combination, a progressive series of changed, usually reduced, turning radii occurs for the outer tire of the virtual front axle for each semitrailer-like unit. By analyzing each semitrailer-like unit in the order it appears in the trailer combination, using in sequence the turning radius of the outer front tire on each real or virtual front axle, it is possible to obtain a series of separate offtracking measurements for each vehicle. These measurements can be added to obtain the peak overall offtracking of the complete trailer combination.

When determining the offtracking of trailer combinations having full trailers, the phenomenon of negative offtracking must be considered. Negative offtracking occurs when the edge of the cargo body opposite the pintle hook swings outside the path of the outside of the outer rear tire on a turn, as shown in figure 9. In effect, negative offtracking increases the turning radius of the following semitrailer-like unit. The magnitude of negative offtracking depends upon the wheelbase of the towing vehicle, the length of rear overhang to the centerline of the pintle hook, turning radius, and the degree of turn. The negative offtracking measurements for practical power vehicles and towing semitrailers are given in tables 1 and 2.

Steering Systems

Ackerman steering

An understanding of different aspects of vehicle turning and offtracking requires information on the systems of steering used on most highway vehicles. Single-unit vehicles, automobiles, light trucks, tractive trucks, and tractors, are equipped with Ackerman-type steering. The Ackerman system was invented in Germany about 1817 and patented by an Englishman in 1818. It is the preferred steering system because it provides better stability to the front end of the vehicle during a turn. In the Ackerman system the two front wheels are mounted on short, stub axles that are connected to the steering kingpins. The kingpins are connected to the front-wheel spring suspension and are supported by the vehicle chassis or sometimes by a rigid beam-type front axle. During a turn, the front wheels are pivoted on the kingpins by the steering linkage and other mechanisms connected to the steering wheel.

Vehicles equipped with Ackerman steering are limited in their offtracking by the minimum turning radius curve that can be followed by the outer front wheel. This minimum turning radius usually is limited, because of mechanical obstructions, by the degree to which the inner front wheel may be turned. This limited turning capability of the inner front wheel is called the cramp angle. On most over-the-road trucks the maximum cramp angle is between 30 and 35 degrees. Recently, however, the manufacturers of city delivery trucks have been widening the distance between front wheels and are obtaining cramp angles of 45 to 50 degrees. Offtracking data for such vehicles are included in the figures in this article.

Fifth-wheel steering

Semitrailers, full trailers, and trailer converter dollies operate with a fifth-wheel, pivot-steering principle that is different from the Ackerman system. As trailers are not operated alone, they do not require the front-end stability required for power vehicles. In the fifth-wheel, pivot-type of steering system the front wheels are mounted at the ends of a rigid one-piece axle. This axle is pivoted about a kingpin mounted above the lateral center of the axle, where it is connected to the trailer body.

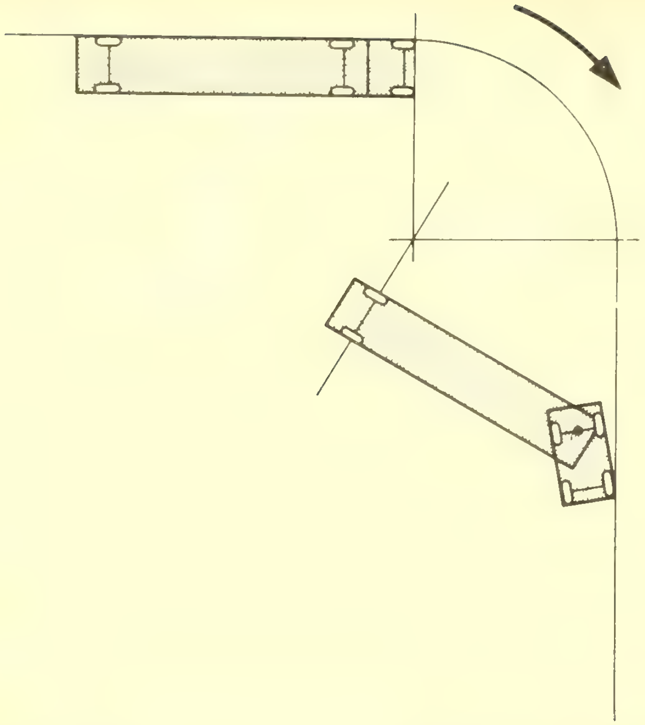

Figure 8.—A long wheelbase combination on a short radius turn, in which the semitrailer passes in back of the turning radius center.

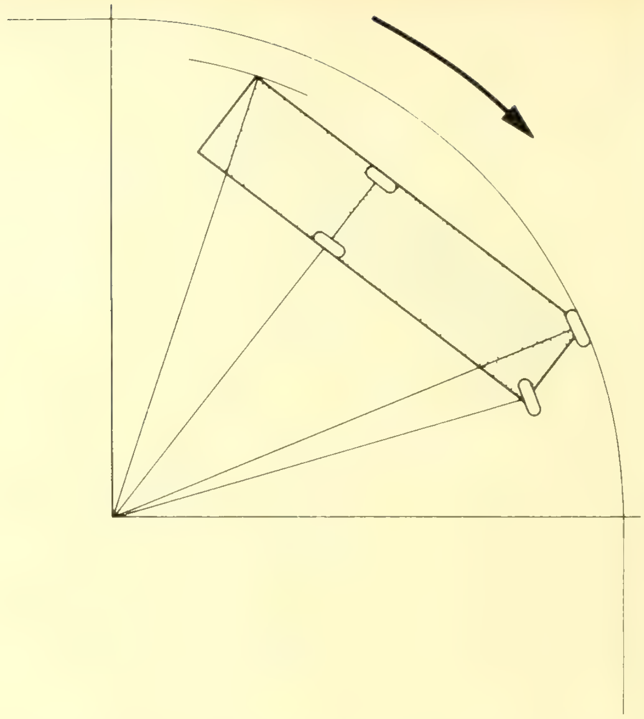

Figure 9.—Negative offtracking in which the path of the outer rear corner of the cargo body has a greater radius than the path of the outer rear wheel.

For semitrailers the rear axle of the tractor acts as the virtual front axle of the trailer. In most designs, the trailer kingpin, surrounded by a lubricated bearing plate, is attached to the underside of the semitrailer, usually about 3 feet back of the front end of the trailer. Another bearing plate, equipped with a kingpin locking device, is mounted on the tractor chassis over the rear axle. This fifth wheel engages and holds the trailer kingpin and allows the trailer to be pulled and steered by the tractor. This system permits easy coupling or uncoupling and the interchanging of trailers.

Full trailers are basically semitrailers that have one or two types of front-axle assemblies; the front axle is permanently attached to the trailer or is removable. The removable, front-axle assemblies are known as trailer converter dollies. They consist of one or more one-piece axles supported by a spring suspension system and have a fifth wheel mounted above the center of the axle. Both the trailer converter dollies and the permanently attached front-axle assemblies have towbars affixed at a 90-degree angle to the axle. The towbar has an eye that engages a vertical pintle hook on the rear of its towing vehicle. Once engaged, the towbar may pivot freely about its pintle hook. Such pivoting is limited only by interferences with rear frame parts of the towing vehicle. Because of the free pivoting action of the towbar, both types of front-axle assemblies of full trailers act as short wheelbase semitrailers in making a turn. Thus a full trailer turns and offtracks in the same manner as a semitrailer connected in tandem to another semitrailer, both of which have fifth-wheel pivot steering.

In the steering and offtracking behavior of full-trailer front-axle assemblies, their virtual front axle can be assumed to be located at the center of the pintle hook. The wheelbase of such devices, therefore, is measured from the center of the towbar eye to the center of the axle. With fifth-wheel-pivot steering no cramp-angle problem occurs and the angular relationship between the towing vehicle and the semitrailer is not restricted; it may be as much as or more than 90 degrees.

Vehicle Models and Instrumentation

The relations on offtracking contained in this article were obtained primarily through the use of scale models of highway vehicles. The models were designed to provide a good simulation of actual vehicle turning characteristics for many different types and lengths of single-unit vehicles and trailer combinations. To expedite the study, models were designed as detachable components that could be quickly assembled or disassembled. The models, equipped with an Ackerman steering mechanism, were constructed to a scale of 0.75 inch equals 1 foot and the width over the tires equals 8 model feet.

The models were operated on a smooth surface of 4-by-8-foot panels placed on a level concrete floor. The panels were assembled in 16-by-16-foot squares and circles were painted from the center to simulate highway curves ranging from a turning radii of 25 to 100 model feet. Radial lines, at 10-degree intervals, and tangents were superimposed upon the test layout, as shown in figure 10. Turning radii of 165 and 225 model feet were obtained by placing 8 additional panels about the original 16-by-16-foot square.

Before each individual test, the vehicle and axle alinement of the model was checked on an 8-foot approach tangent. If the model followed the tangent without any perceptible deviation, it was then guided so that the outside of the outer front wheel followed the circular curve selected for the test. Offtracking tests were conducted with models representing different types of single-unit vehicles and trailer combinations. Included were models of power vehicles with wheel bases ranging from 5 to 30 model feet, tractive truck models with considerable rear overhang, and semitrailer models with wheelbases ranging from 5 to 55 model feet. Full trailer model tests were not conducted as full trailers offtrack the same way as semitrailers.

In the semitrailer model tests the trailer kingpin was positioned directly over the the center of the front axle of a short wheel base tractor model, as shown in figure 11. With the kingpin in this position any offtracking of the tractor did not affect the trailer offtracking; however, the tractor did provide model stability. In all of the model tests the offtracking was measured at the rear or trailing axle of a vehicle or trailer combination. To ascertain the magnitude of negative offtracking on tractive trucks having long rear overhangs, an additional offtracking measurement was taken at the outer rear corner of the model opposite the pintle hook centerline. To expedite the determination of the peak offtracking, a scale was mounted on the vehicle models as shown in figure 11. Offtracking data were obtained by the model assemblies for both 90- and 270-degree turns.

Offtracking Calculations

The results obtained from the tests on vehicle models are shown in figures 5 and 6 for 90 degree turns. They were designed to permit the rapid determination of offtracking for single-unit vehicles and trailer combinations. For single-unit vehicles, the offtracking can be determined directly. Determination of the offtracking for trailer combinations can be obtained by adding together the offtracking of the individual units of the combinations. Semilogarithmic graph paper was used in the preparation of these figures. The ordinate in a logarithmic scale represents offtracking in feet. The logarithmic scale was selected to reduce the height of the ordinates for publication. The abscissa represents the turning radius in feet and it has been presented on an equal interval scale. In each figure, the wheelbase curves were drawn in 5-foot increments.

Vehicle offtracking may be evaluated for turning radii of 25 to 225 feet and for wheelbase lengths of 5 to 55 feet. The 25-foot turning radius represents the shortest radius turn studied with the models. At more than a 225-foot turning radius, the offtracking of single-unit vehicles and trailer combinations approaches the maximum offtracking that can be calculated by the SAE equations (*1*). The approximate limits of the minimum radii of turns possible when an Ackerman-type steering system is employed and when the front wheel cramp angle is 50 degrees are also shown. The following examples explain how the data in these figures can be used.

Single-unit vehicles

The offtracking for single-unit vehicles can be determined directly. For example, the peak offtracking for a 2-axle truck negotiating a turning radius of 70 feet on a 90-degree turn is shown in figure 5. If the 2-axle truck had a wheelbase of 30 feet, the peak offtracking would be 6.4 feet. If the same 2-axle truck was negotiating a 70-foot radius curve through a 270-degree turn, the peak offtracking would be 7.0 feet (fig. 6). If the minimum turning radius for the 2-axle truck is desired, it can be approximated by use of the dashed curve shown on the figure. If the front wheel cramp angle is 50 degrees, a 30-foot wheelbase, single-unit truck cannot negotiate a curve having a turning radius of less than 45 feet (fig. 5).

If the offtracking is desired for a vehicle having a wheelbase between those represented by the wheelbase curves in either figure 5 or 6, figure 12 may be used to interpolate between the wheelbase curves. For example, if offtracking had been desired for a

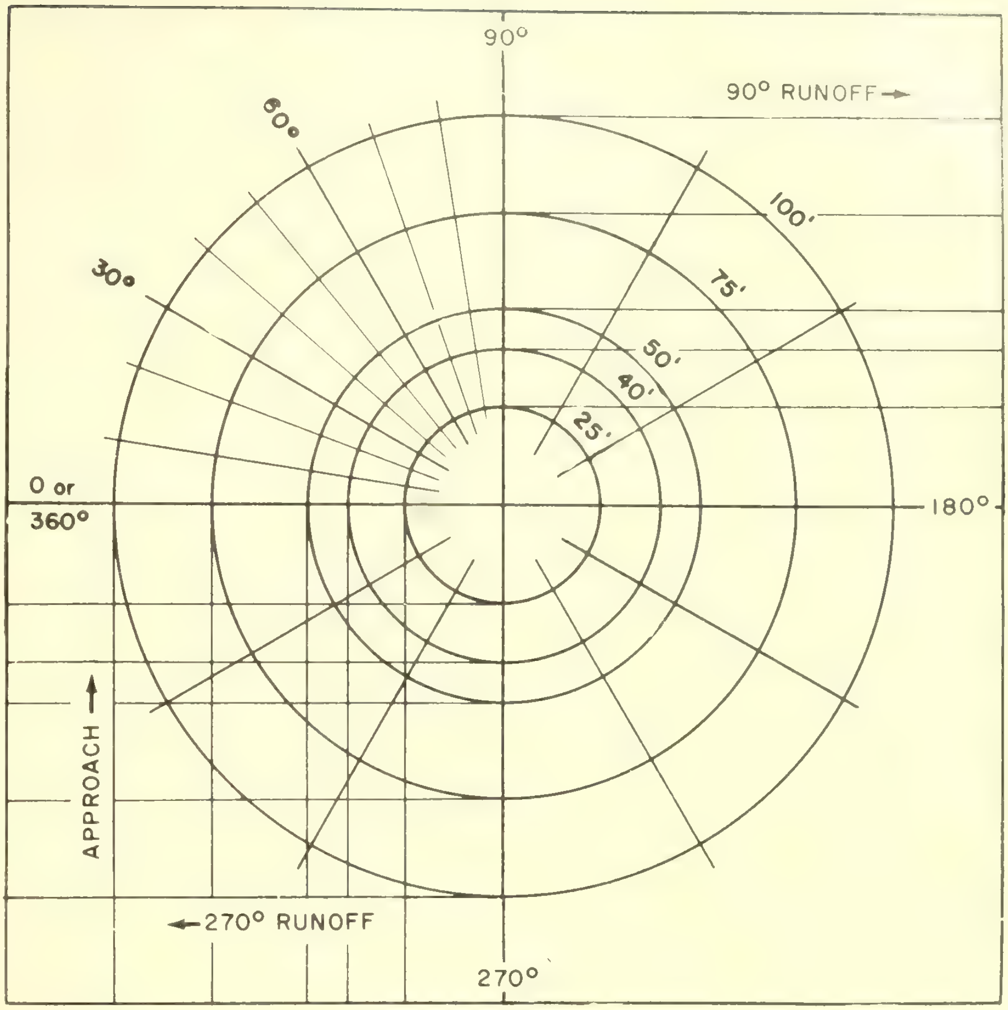

Figure 10.—Schematic arrangement of guidelines on floor panels.

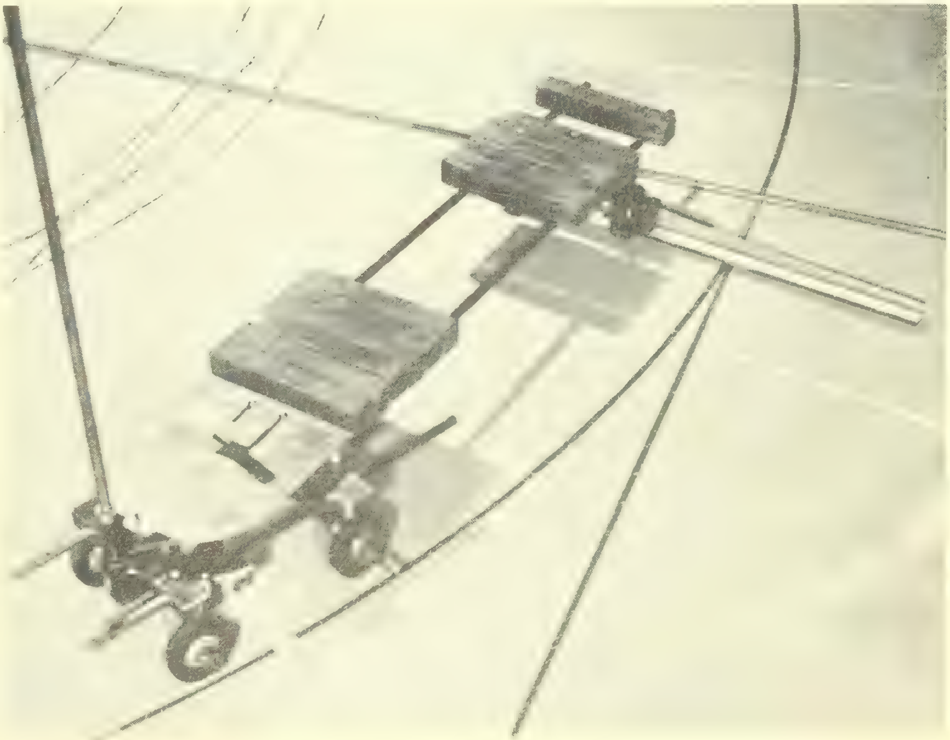

Figure 11.—Semitrailer model and fifth-wheel pivot steering.

Table 1.—Negative offtracking for 90-degree turns

Wheelbase	Turning radius of outside of outer front wheel	Offtracking of outside of outer rear wheel	Negative offtracking of outer rear corner opposite pintle hook for—				
			3-foot overhang	5-foot overhang	7-foot overhang	9-foot overhang	11-foot overhang
Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
10	25	2.00	0.19	0.54	1.04	1.70	---
	30	1.71	.16	.44	.85	1.40	---
	40	1.27	.12	.32	.63	1.03	---
	50	1.01	.00	.26	.50	.82	---
	60	.85	.00	.21	.41	.68	---
	70	.75	.00	.18	.35	.58	---
	80	.67	.00	.16	.31	.51	---
	90	.60	.00	.15	.27	.45	---
	100	.51	.00	.13	.25	.39	---
15	25	3.62	.20	.55	1.11	1.78	2.75
	30	3.24	.17	.46	.88	1.45	2.17
	40	2.70	.12	.33	.64	1.07	1.59
	50	2.39	.00	.26	.51	.84	1.25
	60	2.00	.00	.22	.42	.69	1.03
	70	1.75	.00	.18	.36	.59	.88
	80	1.55	.00	.16	.31	.51	.77
	90	1.38	.00	.14	.28	.46	.68
	100	1.23	.00	.13	.25	.41	.61
20	25	5.43	.22	.62	1.23	1.96	2.97
	30	5.05	.18	.51	.96	1.56	2.33
	40	4.40	.13	.35	.68	1.12	1.66
	50	3.89	.10	.27	.53	.87	1.29
	60	3.45	.00	.22	.43	.71	1.15
	70	3.08	.00	.19	.37	.60	.90
	80	2.72	.00	.16	.32	.52	.78
	90	2.41	.00	.14	.28	.46	.69
	100	2.17	.00	.13	.25	.41	.62
25	25	8.37	.27	.74	1.43	2.28	3.31
	30	7.80	.20	.58	1.08	1.75	2.58
	40	6.80	.14	.37	.73	1.20	1.77
	50	5.93	.10	.28	.55	.91	1.35
	60	5.28	.00	.23	.45	.74	1.09
	70	4.61	.00	.19	.37	.62	.92
	80	4.10	.00	.16	.32	.53	.79
	90	3.70	.00	.14	.28	.47	.70
	100	3.34	.00	.13	.25	.42	.62
30	25	11.71	.33	.92	1.76	2.73	3.96
	30	10.96	.23	.66	1.24	2.01	2.94
	40	9.50	.15	.41	.79	1.30	1.92
	50	8.30	.11	.30	.58	.96	1.43
	60	7.27	.00	.24	.46	.76	1.14
	70	6.41	.00	.20	.38	.63	.94
	80	5.70	.00	.17	.33	.55	.81
	90	5.11	.00	.15	.29	.48	.71
	100	4.62	.00	.13	.26	.42	.63
35	25	15.08	.44	1.19	2.22	3.47	4.89
	30	14.20	.28	.77	1.48	2.38	3.45
	40	12.55	.16	.46	.88	1.44	2.12
	50	11.10	.12	.32	.62	1.04	1.53
	60	9.60	.00	.25	.48	.80	1.19
	70	8.49	.00	.20	.40	.65	.98
	80	7.52	.00	.17	.38	.56	.83
	90	6.78	.00	.15	.29	.49	.72
	100	6.10	.00	.13	.26	.43	.64

Table 2.—Negative offtracking for 270-degree turns

Wheelbase	Turning radius of outside of outer front wheel	Offtracking of outside of outer rear wheel	Negative offtracking of outer rear corner opposite pintle hook for—				
			3-foot overhang	5-foot overhang	7-foot overhang	9-foot overhang	11-foot overhang
Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
10	25	2.58	0.20	0.55	1.07	1.74	---
	30	1.90	.16	.44	.86	1.41	---
	40	1.27	.12	.32	.63	1.03	---
	50	1.01	.00	.25	.50	.82	---
	60	.85	.00	.21	.41	.68	---
	70	.75	.00	.18	.35	.58	---
	80	.67	.00	.16	.31	.51	---
	90	.60	.00	.14	.27	.45	---
	100	.54	.00	.13	.25	.41	---
15	25	10.40	.31	.83	1.59	2.55	3.68
	30	4.60	.18	.49	.95	1.55	2.28
	40	3.43	.12	.33	.67	1.09	1.63
	50	2.68	.00	.26	.51	.85	1.26
	60	2.18	.00	.22	.42	.70	1.03
	70	1.83	.00	.18	.36	.59	.88
	80	1.61	.00	.16	.31	.51	.77
	90	1.38	.00	.14	.28	.46	.68
	100	1.23	.00	.13	.25	.41	.61
20	25	18.93	.70	1.79	3.19	4.79	4.49
	30	8.70	.21	.58	1.12	1.82	2.67
	40	6.24	.13	.37	.72	1.18	1.75
	50	4.72	.10	.27	.54	.89	1.30
	60	3.71	.00	.22	.43	.71	1.06
	70	3.10	.00	.19	.37	.60	.90
	80	2.73	.00	.16	.32	.52	.78
	90	2.41	.00	.14	.28	.46	.69
	100	2.17	.00	.13	.25	.41	.62
25	25	---	---	---	---	---	---
	30	14.25	.28	.77	1.55	2.39	3.46
	40	9.70	.15	.41	.80	1.31	1.93
	50	7.27	.11	.29	.57	.95	1.39
	60	5.80	.00	.23	.45	.74	1.10
	70	4.81	.00	.19	.37	.62	.92
	80	4.17	.00	.16	.32	.53	.79
	90	3.70	.00	.14	.28	.47	.70
	100	3.34	.00	.13	.25	.42	.62
30	25		---		---		---
	30	26.01	1.00	1.80	4.07	5.85	7.71
	40	14.85	.18	.49	.88	1.52	2.20
	50	11.21	.12	.32	.63	1.03	1.53
	60	8.67	.00	.24	.48	.78	1.17
	70	7.18	.00	.20	.39	.64	.96
	80	6.07	.00	.17	.33	.55	.81
	90	5.26	.00	.15	.29	.48	.71
	100	4.70	.00	.13	.26	.42	.63
35	25	---	---	---	---	---	---
	30	38.00		---	---		---
	40	18.80	.20	.58	1.13	1.83	2.68
	50	15.65	.13	.36	.71	1.16	1.72
	60	12.50	.00	.26	.51	.85	1.26
	70	10.15	.00	.21	.41	.67	1.00
	80	8.50	.00	.17	.34	.56	.84
	90	7.35	.00	.15	.30	.49	.73
	100	6.57	.00	.13	.26	.43	.65

single-unit truck having a 11-foot wheelbase the following procedure would be employed. Use the vertical distance from figure 5 or figure 6 between the 10- and 15-foot wheelbase curves and locate the same distance vertically on figure 12 between the 10- and 15-foot lines. At this location, the vertical distance between the 10- and 11-foot lines is then carried back to figure 5 or 6 and located vertically above the 10-foot wheelbase curve; the offtracking is then read on the ordinate horizontally opposite the point representing the 11-foot wheelbase. When negotiating a 90-degree turn, the peak offtracking for this single-unit vehicle would be 0.9 foot.

Tractor combinations

Offtracking is determined for tractor semitrailers by adding the offtracking data for the individual vehicles of the combinations. For example, the peak offtracking for a 2–S2 combination negotiating a turning radius of 100 feet through a 270-degree turn would be determined from figure 6. The dimensions of the sample 2–S2 trailer combination are given in figure 13. The peak offtracking is determined for the tractor having a 10-foot wheelbase (0.54 foot, fig. 6). To determine the semitrailer offtracking, its turning radius and wheelbase must be known. The assumption has been made that the kingpin is located directly above the centerline of the rear axle of the tractor (figs. 5 and 6). In effect, the rear axle of the tractor becomes the virtual front axle of the semitrailer. The semitrailer turning radius is computed by subtracting the tractor offtracking from the tracking turning radius; it is 99.5 feet. The semitrailer wheelbase is the distance from the kingpin to the centerline of the rear axle on the semitrailer. In this example, the semitrailer had a tandem rear axle, therefore, the wheelbase of 29 feet is the distance from its kingpin to the centerline between the tandem axles. Reference to figure 6 shows the semitrailers peak offtracking was 4.4 feet when the turning radius was 99.5 feet and the wheelbase was 29 feet. The offtracking for the tractor-semitrailer portion of the trailer combination is the sum of the tractor offtracking and the semitrailer offtracking (0.54 plus 4.4 or 4.94 feet). As previously determined, the tractor semitrailers peak offtracking, in reference to the centerline between the tandem axles, was 4.94 feet when negotiating a 100-foot turning radius through 270 degrees. But the offtracking for the trailer converter dolly and the full trailer also must be determined.

The trailer converter dolly is connected to the semitrailer at a pintle hook, located 7 feet behind the centerline between the tandem axles. Negative offtracking is present in that the path of the outer rear corner of the semitrailer swings outward from the turning radius center. The magnitude of negative offtracking can be determined from table 1 or 2. With a wheelbase of approximately 30 feet, a turning radius of nearly 100 feet, and a 7-foot rear overhang to the pintle hook, the

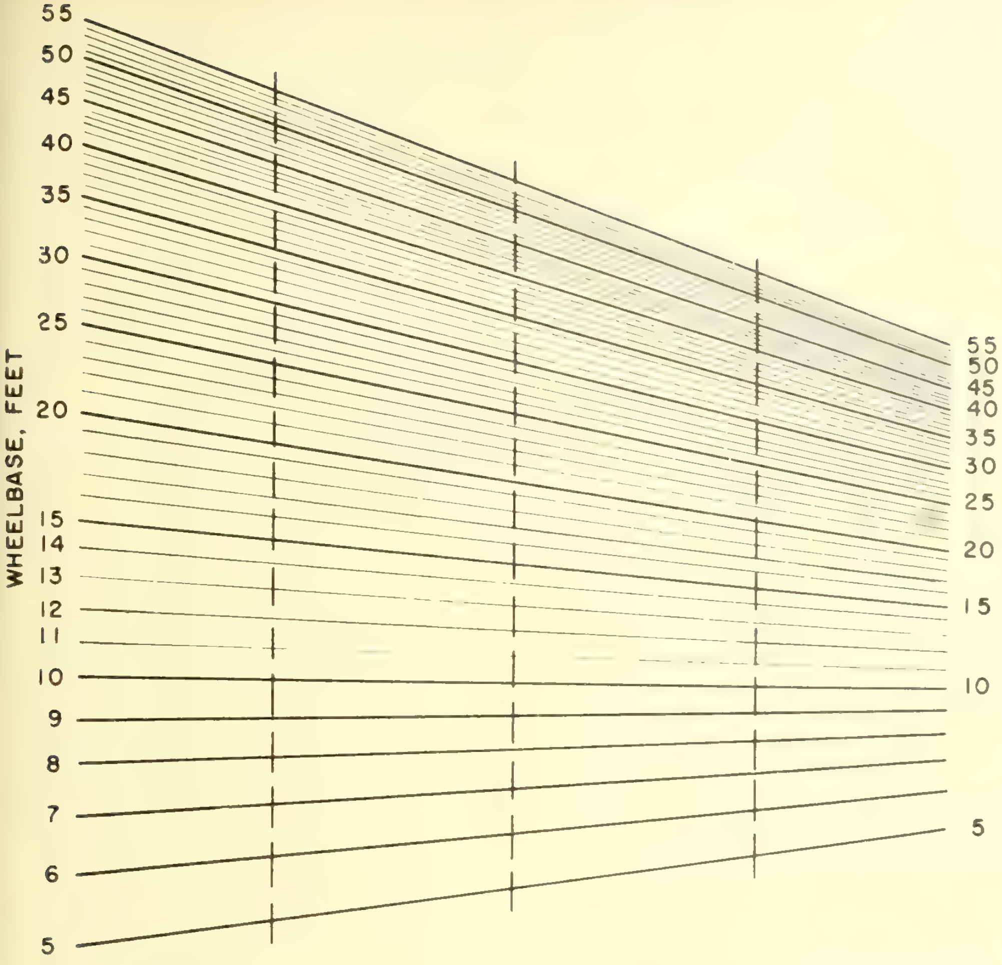

Figure 12.—Interpolation guide for wheelbase lengths between 5-foot interval wheelbase curves.

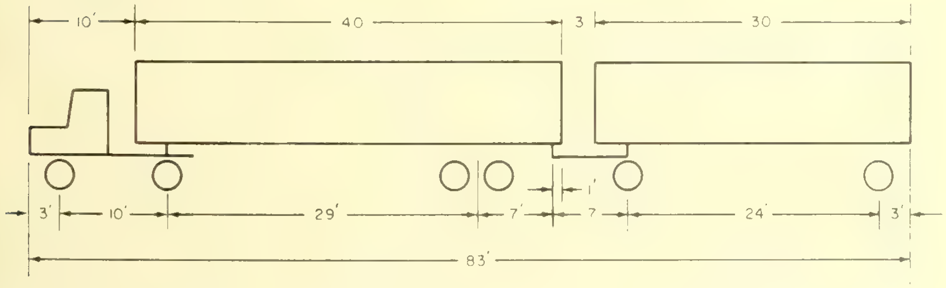

Figure 13.—Dimensions of trailer combination used to demonstrate calculations.

negative offtracking of the virtual front axle of the dolly is 0.26 foot for a 270-degree turn.

The pintle hook was assumed to be in the center of the virtual front axle of the trailer converter dolly. The trailer converter dolly turning radius is found by subtracting the tractor semitrailer offtracking (4.94 feet) from the turning radius of the tractor and adding to that the negative offtracking. Thus, the turning radius of the virtual front axle of the trailer converter dolly would be 95.32 feet (100.00 minus 4.94 plus 0.26). With the trailer converter dolly having a wheelbase of 7 feet and a turning radius of 95.32 feet, the dolly offtracking of 0.28 foot, was determined from figure 6.

After the peak offtracking of the trailer converter dolly is obtained, the offtracking for the full trailer is determined. The turning radius for the full trailer is computed in the same way as for the semitrailer. Thus, the turning radius of the virtual front axle of the full trailer is 95.04 feet (95.32 minus 0.28). The kingpin on the full trailer is assumed to be directly above the centerline of the dolly axle. If a tandem axle dolly had been used, the kingpin would be located directly above the centerline between the tandem axles. In effect the dolly axle is the virtual front axle of the full trailer. For a full trailer having wheelbase of 24 feet and a turning radius of 95.04 feet, the full trailer offtracking is 3.2 feet (fig. 6).

The offtracking of the entire 2–S2–2 tractor semitrailer and full trailer with an overall length of 83 feet is the sum of the offtracking of the individual vehicles minus the negative offtracking. The peak offtracking for the 2–S2–2 combination example would be 8.16 feet (0.54+4.40+0.28+3.20−0.26) when negotiating a 100-foot turning radius curve through a 270-degree turn. The turning track width would be 16.16 feet (8.16+8.00). The inside curb radius is equal to the turning radius minus the turning track width or 83.84 feet (100−16.16). The computed SAE maximum offtracking for this vehicle is 8.42 feet.

Truck and full trailers

The peak offtracking for truck and full trailers also can be determined by use of data shown in figures 5 and 6. The same techniques are used for determining the offtracking of the individual vehicles of a truck and full trailer combination as are used for determining the offtracking of a tractor semitrailer and full trailer combination.

Offtracking Comparisons

To illustrate that different types and sizes of vehicle combinations offtrack differently, several representative long trailer combinations were selected for comparisons. Dimensions of the trailer combinations are listed in table 3 and offtracking characteristics are listed in table 4. In table 3, the 2–S1, 2–S2, and 3–S2 combinations have over-all lengths shorter than either of the 2–S1–2 combinations listed. However, the 65-foot, 2–S1–2 combination offtracks less than either of the tractor-semitrailer combinations and the 71-foot 2–S1–2 combination has approximately the same offtracking as the tractor semitrailers. Because vehicles do offtrack differently; highway design engineers use as guides the highway design vehicles recommended by the American Association of State Highway Officials. The offtracking characteristics given for vehicles in the 1965 proposed revision of the AASHO highway design vehicles also are shown in table 4. The dimensions proposed for these design vehicles are given in table 5.

Model and SAE Offtracking Comparisons

Offtracking results computed from tests with the models were compared to results obtained from the SAE offtracking equations. Comparisons were made for 90- and 270-degree turns on 50- and 150-foot turning radii, respectively. Most of the vehicle models obtained their maximum offtracking prior to reaching the 270-degree exit tangent, therefore, the results could be validated by com-

Table 3.—Dimensions of some of the trailer combinations listed in table 4

	2-S1	2-S2	3-S2	2-S1-2	2-S1-2	3-S2-4
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Length of each trailer	40	40	40	27	30	40
Front bumper to nose of first trailer	10	15	15	8	8	16
Space between trailers				3	3	3
Width over tires	8	8	8	8	8	8
Wheel base, tractor (to centerline of tandem axle	10	15	15	8	8	16
Front bumper to front axle of tractor	3	3	3	3	3	3
Wheelbase, semitrailer	31	29	32	21	21	32
Rear pintle hook overhang of semitrailer				3	3	5
Wheelbase of trailer converter dolly				6	6	6
Wheelbase, full trailer				21	21	32
Rear overhang of trailer	3	8	5	3	3	5
Overall length of trailer combinations	50	55	55	65	71	99

Table 4.—Vehicle offtracking computations and AASHO proposals

Vehicle types	Overall length	90-degree turn, 50-foot turning radius			270-degree turn, 150-foot turning radius		
		Offtracking	Turning track width	Inside curb radius	Offtracking	Turning track width	Inside curb radius
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Long trailer combinations:							
2-S1	50	11.3	19.3	30.7	4.5	12.5	137.5
2-S2	55	10.3	18.3	31.7	3.7	11.7	138.3
3-S2	55	11.7	19.7	30.3	4.3	12.3	137.7
2-S1-2	65	9.4	17.4	32.6	3.3	11.3	138.7
2-S1-2	71	12.5	20.5	30.5	4.4	12.4	137.6
3-S2-4	99	22.0	30.0	20.0	8.1	16.1	133.9
AASHO proposals:[1]							
Passenger cars	19	1.1	7.1	42.9	0.4	6.4	143.6
Other vehicles:							
2	30	3.8	12.3	37.7	1.2	9.7	140.3
2-S2	50	7.8	16.3	33.7	2.7	11.2	138.8
3-S2	55	11.8	20.3	29.7	4.2	12.7	137.3

[1] Proposed 1965 revision of AASHO highway design vehicles.

Table 5.—Dimensions in proposed 1965 revision of AASHO highway design vehicles listed in table 4

	Single-unit truck or bus	2-S2 trailer combinations, WB-40[1]	3-S2 trailer combinations, WB-50[1]
	Feet	*Feet*	*Feet*
Length of trailer		36	37
Front bumper to nose of trailer		14	18
Width over tires	8.5	8.5	8.5
Wheelbase, single-unit truck, or tractor	20	13	18
Front bumper to front axle	4	4	3
Wheelbase, semitrailer		25	30
Rear overhang	6	8	4
Overall length of vehicles	30	50	55

[1] AASHO identification for trailer combinations by wheelbase.

Table 6.—Model and SAE offtracking test results

Trailer combinations	Trailer length	Overall length	Offtracking			
			90-degree turn, 50-foot turning radius		270-degree turn, 150-foot turning radius	
			Model	SAE	Model	SAE
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
2-S1	40	50	11.30	16.62	4.47	4.36
2-S1	40	55	13.00	18.75	4.96	4.81
2-S2	40	50	8.90	11.69	3.28	3.26
2-S1	40	55	10.30	13.51	3.72	3.69
3-S2	40	50	10.00	14.46	3.87	3.90
3-S2	40	55	11.65	16.45	4.30	4.34
2-S1-2	2×27	65	9.38		3.28	3.06
2-S1-2	2×30	71	12.48		4.41	4.32
3-S2-4	2×40	99	21.97		8.12	8.16

paring them with the maximum offtracking results computed by the SAE equations. Results of some of the comparisons are listed in table 6. As shown the tractor-semitrailer models negotiating the 90-degree turns on the 50-foot turning radius curve did not obtain SAE maximum offtracking. For the tractor-semitrailer and full trailer models negotiating the same turns, the SAE offtracking equation is not applicable because the trailing rear axle of the trailer combination passed behind the turning center.

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Test of Some First Generation Residential Land Use Models

BY THE
OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

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The relative accuracy of forecasting land use by five residential land use simulation models was tested in the study discussed in this article. Explicit model statements and descriptions of the procedures used in calibrating the forecasts are given. All conditions for the test models were held constant except the interrelations among variables, so that differences among the forecasts would be caused only by inherent differences in the models themselves. Results of the study reported here cannot be considered definitive because of the differences in amount of information required and the differences in the degree to which the models were fitted to data. Also, the study represents only one of many possible test conditions.

Some conclusions useful to small metropolitan areas, lacking resources for development of new models and refinement of existing models, can be drawn from the study results. Simple, nonbehavioral residential land use forecasting models were shown to be sufficiently accurate for metropolitan areas of 100,000 population or more. The models also were effective in forecasting for geographic units having a population of about 2,000, although results for smaller areas were not accurate. In the evaluation none of the five forecasting models was superior; use of any one of the models for urban planning would be preferable to forecasting without a model.

Introduction

A COMPARATIVE evaluation of five operational residential land use forecasting techniques, four of which have been used previously in urban transportation planning studies, is discussed in this article. The techniques are representative of the earliest efforts in the development of operational urban activity simulation models and serve, either in the original or modified form, many transportation planning organizations. The more complex and accurate urban activity simulation models, currently under development, often draw upon fundamental concepts that originated with or were adapted to early techniques. Improvements being introduced in these second generation models include more complex statistical estimating procedures, the stratification of residential locators into distinct groups, and the incorporation of behavioral relationships in model formulation. Although newer techniques may require several years of research, evaluation, and refinement before they become fully operational, the less sophisticated techniques evaluated here would be useful to small metropolitan areas lacking resources for developmental research.

The primary objective of the research discussed in this article was to compare the relative accuracy of the existing techniques through a series of after the fact tests, in which all conditions were constant except the interrelationships among variables, so that differences in forecasts would be a function only of inherent differences in models. However, the results of this study cannot be interpreted as definitive statements for several reasons. First, the tests are not all of the same class. Some are forecasts, others data fitting problems that involve fitting different numbers of parameters. Second, more information is used in some than in others. Third, the results represent a sample of one of a large universe of possible test conditions; entirely different results might be obtained if these models were used on different data in other cities, for different time periods, by other forecasters.

Conclusions and Findings

- Simple, nonbehavioral residential land use forecasting models are sufficiently accurate for use in small metropolitan areas having populations of 100,000 or more. The analysis of the spatial structure and growth pattern of Greensboro, N.C., confirms the belief that analytical planning would be helpful in the development of such urban areas.
- Land use forecasting with simple first generation models produced reasonably accurate results for levels of geographic organization where the average area unit contained a population of about 2,000. Efforts to forecast growth for much smaller areas may prove unjustified. At zone levels of about 300 population, these models appeared to offer little or no assistance in forecasting land use.
- Differences in accuracy among the five forecasting models were not large enough to warrant a strong recommendation for any one model in preference to another. Use of any one of the models seems preferable to forecasting land use without analytical techniques.
- The accessibility model produced the most accurate forecast, of methods used, without calibration to time series data. Errors in fitting were insensitive to small changes in the exponent of accessibility.
- Although five or more parameters were used in the regression model equations, none of the multiple linear regression models were better than the two variable fitted models.
- Multiple regression models possess drawbacks. If the dependent variable is expressed as an extensive quantity, such as an increase in dwelling units, measured relations with independent variables will be influenced by peculiarities of area definition and size, and they might not conform to logical hypotheses regarding the land development process. Nonlinear transformations on the dependent variable, such as logarithms or fractional power functions, are unsatisfactory. The use of least squares calculations tends to bias the parameter estimates to produce good fits to small independent variables and poor fits to large ones. Expression of the dependent variable as an intensive quantity, such as dwelling unit increase per unit of geographic area, may be the most satisfactory operational solution except that relations actually nonlinear may not be represented properly. This might be corrected by treating certain independent variables as sets of dummy variables.
- Although the two intervening opportunities models performed satisfactorily, possible improvement might be obtained by allocating growth from all major centers of employment rather than from a single point, the CBD. In addition, each of the two models plots as a straight line plot on different semilogarithimic coordinates. This linear relationship was found not to hold true for the entire study area of Greensboro. Apparently the hypotheses are valid, but separate functions may be necessary for the established inner city area and the developing suburban area.

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- The C.A.T.S. forecasting model differs from the other models: It forces the analyst to become intimately familiar with the study area before attempting to make a forecast. This is a strong feature. The graphic analyses that the method is based on represent descriptions of the key spatial relations of a metropolitan area, even for small areas. The methods of analysis are useful regardless of the forecasting technique used. They can serve as a check on the reasonableness of forecasts made by less subjective models.

However, as applied in the study reported in this article, the method is time-consuming; it requires considerable handwork and far more data manipulation. The method is less adaptable to the computer and would be cumbersome for testing alternative land use policies or for recursive use in combination with submodels.

- The five models tested are not representative of the type and number of current land use forecasting models. They represent the initial models but lack the sophistication and elegance of more recent model developments. These are descriptive models and do not involve either the behavior of decision makers or have any theoretical content. It is highly probable that the key to increased forecasting accuracy for small subareas is in the ability of the analyst to simulate the decision process for subpopulations of the area.

General Procedures

The five residential land use forecasting procedures are each variants of work done by others. The only innovations introduced during the study were simplifications and modifications made for particular conditions. In any realistic planning application, more care would have to be given to the particular forecasting tool used. Trends would be more carefully analyzed, the forecasters would be more familiar with the area being studied, and output of models would be scrutinized and modified as judgment indicated. In the research reported here the models were applied crudely and the immediate output was accepted. The techniques were: (1) The density-saturation gradient model; (2) accessibility model; (3) regression model; and (4) two intervening opportunities models.

The density saturation gradient model (DSGM) is a simplification of the approach used for the *Chicago Area Transportation Study* (C.A.T.S.) (*1*).[2] Of the five models, the DSGM is the least computer oriented, it is more demanding of subjective inputs, and, therefore, it is the least suitable for objective comparison with other forecasting procedures. The DSGM method is based on the regularity of the decline in density and percentage of saturation with distance from the central business district (CBD) and the stability of these relations through time.

The accessibility model is based upon a concept formulated by Walter Hansen (*2*). Growth in a particular area is hypothesized to be related to two factors: The accessibility of the area to some regional activity distribution, and the amount of land available in the area for development. The accessibility of an area is an index representing the closeness of the area to all other activity in the region. All areas compete for the aggregate growth and share in the growth in proportion to their comparative accessibility positions weighted by their capability to accommodate development as measured by vacant, usable land.

The multiple linear regression model is a popular approach because of its operational simplicity and because several variables can be incorporated (*3*). The proportion of total regional growth in a particular area is assumed to be related to the magnitude of several variables that in some manner are measures of geographic desirability. The procedure is to determine those variables and their weights that in linear combination can be related to the growth observed over a past time period. These independent variables and their weights (regression coefficients), in linear combination (the regression equation), then can be applied to the individual analysis areas to forecast the magnitude of growth, which is the dependent variable.

Although more commonly applied to the problem of trip distribution, the intervening opportunities models can be used to simulate the distribution of urban activity. Two separate and distinct models were applied in this study, based upon the general idea that the probability of an opportunity being accepted decreases as some function of the number of opportunities ranked closer to a central distributing point. The Stouffer formulation was originally applied to intra-urban migration (*4*). A related formulation has more recently been investigated as a trip distribution technique (*5*). Schneider's formulation was originally applied to trip distribution (*6*) and is currently being used in distributing urban activity(7).

The test area used for all the work in the study reported here was Greensboro, N.C. This city was chosen primarily because a rather extensive information file on a small area basis for two time periods (1948 and 1960) was available. Also, it was believed that Greensboro was representative of the kind and size city for which forecasting techniques of the kind being examined would still be appropriate after the development of more sophisticated models in the large metropolitan areas.

The data utilized in the study were made available by Prof. F. S. Chapin, Jr., of the University of North Carolina and Mr. A. M. Voorhees of Alan M. Voorhees and Associates. The data obtained from the University of North Carolina contained information for the Greensboro area coded to 3,980 grid cells, each one a 1,000 feet square, for a circular area of about a 7-mile radius. These data included quantitative measures of land use; population; residential density; proximity to different activities and the CBD; and some environmental measures that are listed completely in a monograph on *Factors Influencing Land Development* by F. S. Chapin, Jr., and S. F. Weiss, University of North Carolina, April 1962. Most of these data were available at the grid level for two time periods, 1948 and 1960.

The data supplied by Alan M. Voorhees and Associates included 1960 population, employment, accessibility to shopping, and accessibility to employment, for about 250 zones. Accessibility measures were computed from zone-to-zone traveltimes over the highway network.

Problems encountered

Problems were encountered in combining the data from these two sources in a form suitable for testing the models. A principal difficulty was the aggregation of grids to zones. Because it was desirable to work at a level of aggregation more typical of transportation studies, new zone boundaries following grid lines approximating the irregular old zone boundaries had to be defined. No important error was introduced as only accessibility scores from the original zone file were used in subsequent analyses. All extensive quantities used were grid aggregates.

Consideration of all data sources and the purpose of the study led to the decision to use dwelling units as the items to be predicted. However, another problem was encountered as data were not directly available. Estimates were made and different checks applied by using the 1948 land area, the 1948 USGS map for suburban areas, the 1950 Census block statistics for the central city and the 1960 land area and dwelling unit densities.

A third problem was the estimation of accessibility measures for 1960 for certain zones at the fringe. The area covered by the zone file did not extend to the boundaries of the grid coverage area in all directions. Rather than eliminate this area entirely, estimates of accessibility measures were made for about half of the outer ring of zones by examining contours of isoaccessibility lines, which follow fairly regular patterns in the fringe areas.

Model Description And Methodology

Explicit model statements and descriptions of the procedures used in calibrating or forecasting for each of the five methods are discussed here.

Density-saturation gradient model

The DSGM is the least formally structured forecasting procedure of the five. No formal theoretical statements or mathematical hypotheses are required, although the staff of the C.A.T.S. has presented excellent conceptual explanations of its empirical findings and rationale for their projections (*1*). Theoretical development, however, was not essential to the purpose of the research reported in this article.

Certain limitations for application of the DSGM to the Greensboro area existed prior to the testing. The only known previous application of this technique was for the Chicago area. Some initial fear existed that

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[2] References identified by italic numbers in parentheses are listed on page 109.

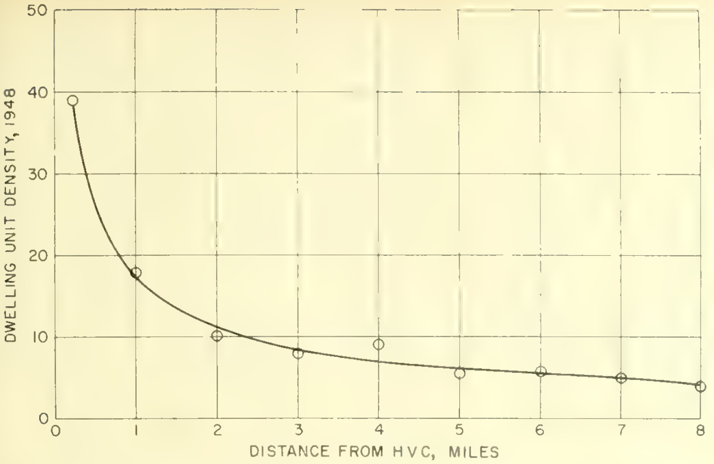

Figure 1.—Dwelling unit density by distance bands for 1948.

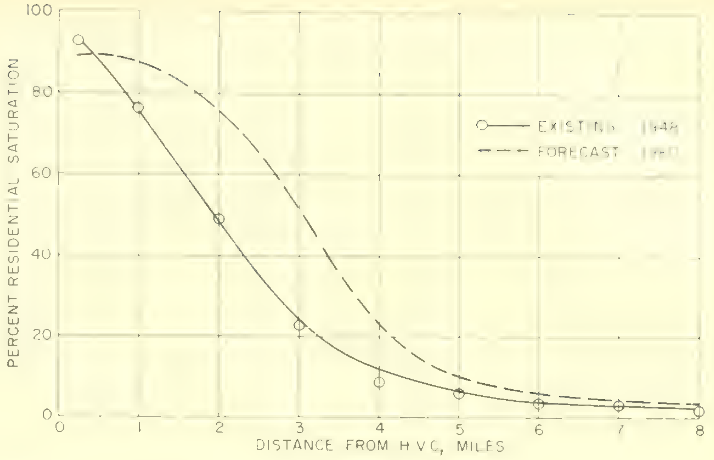

Figure 2.—1960 forecast of percentage of residential saturation by distance bands.

the regularities in activity distribution around the central place, which is axiomatic to the method, would not be manifest for a city the size of Greensboro. The declines in density and percent capacity are the result of the operation of the competitive land market, a mechanism that might not exert a dominating influence upon spatial organization in a city of Greensboro's size. However, these fears were unwarranted and, in fact, the distribution of residential activity was markedly structured around the CBD.

Two trials

Two semi-independent forecast trials by the DSGM were made to determine the sensitivity of the results to variations in the critical assumptions made. The forecasts were distinct in that the first trial was made using airline distance from the high value corner (HVC) as the key spatial variable. The HVC is a point representative of the hypothetical activity center of the CBD. Traveltime to the HVC was used in the second trial.

The relation between the 1948 dwelling unit density and the airline distance from the HVC are shown by data in figure 1. Each point on the plot represents the gross residential density (street area included) for a ring around the HVC. With the exception of the first or CBD ring, each ring was determined by the boundaries of zones whose centroids fell within plus or minus one-half mile of the nominal distance of the ring from the HVC. The plot of the data shows a decline in residential densities with distance from downtown Greensboro in 1948. This decline was encouraging because the reliability of the DSGM depends on the strength and stability of this relation.

The DSGM depends equally upon the correlation between distance and percent saturation. To compute the latter, residential capacity must be defined. Capacity is defined mathematically as existing dwelling units plus the product of vacant available, suitable land, and expected residential density. A decision had to be made as to the density values to be used in the computation. Theoretically the density values should be the anticipated average density at which all future residential development will occur. Thus density should be developed from an intensive analysis of trends in residential density patterns and zoning policies. For purposes of the study, however, future densities for each zone were assumed to be those shown by the smooth hand-fitted curve developed for figure 1. Before acceptance of density data shown by this single curve, gradients were plotted for each of five sectors. Although this plotting showed less than regular relationships, no significant variation in density gradients was noted between sectors.

Figures for vacant, suitable land for residential development were estimated by subtracting those for marginal land and land zoned for nonresidential uses from the amount of 1948 nonurban land. A systematic, but subjective procedure was used in the treatment of zoning: Land use was weighted by factors ranging from zero for grids zoned only for industrial use to 1.0 for grids zoned only for residential use; land use in grids zoned for mixed uses and other nonindustrial uses was weighted subjectively on a scale from zero to unity.

Saturation density

Having computed future residential development densities and vacant available land, it was possible to compute both the residential saturations in dwelling units and the existing percentage of saturation for each distance ring from the HVC. The resultant percentage from the division of saturation into 1948 dwelling units was used to construct the percentage of saturation gradient. The residential saturation plot, shown in figure 2, conforms to that expected for an urban area. The distinct and sharp transition between the 3½- and 4½-mile points indicates a transition in the character of the area used for the study from urban to mostly rural. The almost negligible slope of the curve beyond the 4½-mile point indicates agricultural development and the absence of strong competition for location close to central Greensboro.

The 1960 projection of the percentage of residential saturation is also shown in figure 2. The 1948 percentages of saturation gradients also were plotted by sector; as for the density gradient some scattering of points was noticeable but this scattering was not sufficient to require using sector specific gradients. The 1960 projection is the most critical and subjective step in the forecasting process. The only restraint used in developing the projected curve was to assure that the area under the curve would reflect the projected regional growth. The number of dwelling units in the study area increased from 27,191 to 41,250 in the 1948 to 1960 period, a growth of 52 percent.

Many different procedures can be used to establish an acceptable projection of the percentage of saturation gradient. It was, however, considered useful to develop the overall scale—the area under the final curve that would be commensurate with the requirement for projection of the regional population. As a first approximation, each ordinate rate was raised a distance equivalent to 52 percent of the 1948 rate. The resultant curve then approximated the forecast condition under the assumption of uniform growth over the entire region. General criteria were then introduced to modify the first approximation of the shape of the gradient in 1960. These were that: (1) The bulk of the residential growth would occur in the 2-, 3-, and 4-mile rings; (2) the inner ring would suffer a slight decline; (3) the shape of the gradient would tend to bow out in the 1- to 3-mile range; (4) the sharp transition in slope of the 1948 saturation gradient, observed at about the 4- to 5-mile point, would become less abrupt for 1960; and (5) the land use for areas 5 miles and beyond would show some exurban growth, but the general flat slope would remain.

Dwelling unit growth

Relatively few attempts were necessary before a solution was obtained that was of satisfactory shape and that conformed to the actual 1948–60 increase in total dwelling units. The forecast dwelling unit totals by analysis

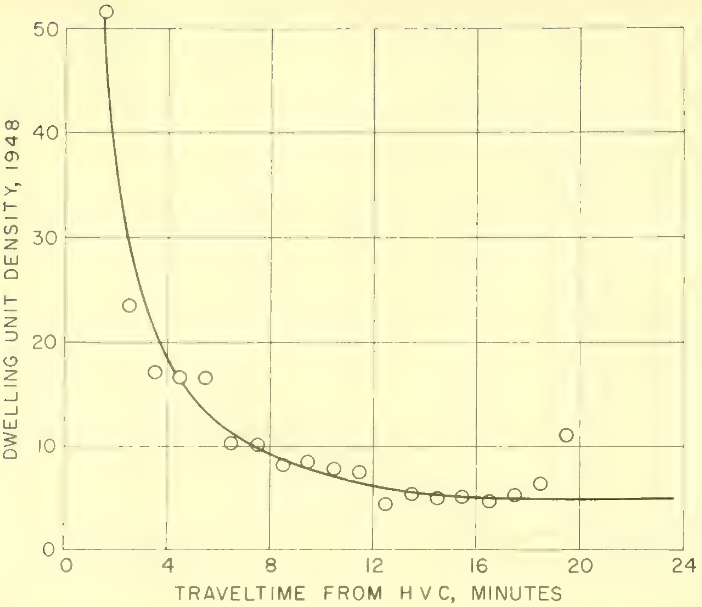

Figure 3.—Dwelling unit density by time bands for 1948.

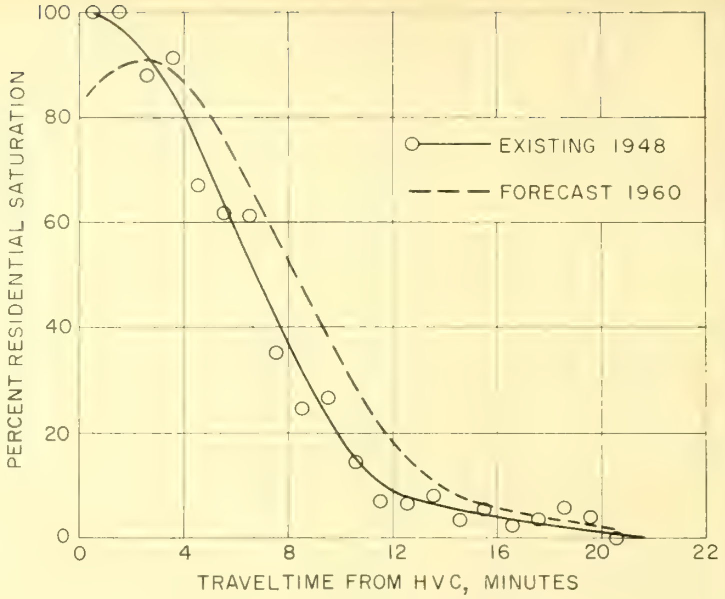

Figure 4.—1960 forecast of percentage of residential saturation by time bands.

ring were established by multiplying the appropriate ordinate figure (fig. 2) from the forecast percentage of saturation gradient by the ring saturation quantities.

The projected growth of each ring was distributed to zones in a two-step process following the logic of the C.A.T.S. The distribution of projected growth to districts (defined by ring-sector boundaries) was handicapped by a lack of historical data. Ideally the trends in land use composition and growth rates between sectors should be studied in detail. For trial one, however, the simple assumption was made that sectors would share growth in proportion to available residential capacity. The final distribution to zones was based on a systematic but subjective linear weighting of the following factors: (1) Distance to convenience shopping; (2) available residential capacity; (3) distance to the major street system; (4) percentage of industrial development in the zone; and (5) percentage of residential development in the zone.

Trial two was conducted independently of trial one and differed from it in two ways. First, traveltime to the HVC was substituted for airline distance as the major independent variable. Zones were grouped into 1-minute interval rings for all analyses. Second, ring growth was allocated to sectors—that is, the district-level forecast—in proportion to the product of each sector's available residential capacity and the number of existing (1948) dwelling units. The same processes for trial two as for trial one were used in estimating density and holding capacity, in establishing sector definitions, and in allocating growth from districts to zones.

Figure 3 shows the dwelling unit density gradient as determined from the ring analysis in trial two. The general shape of the curve is the same as that for trial one. The curves for percentage of saturation of dwelling unit density calculated for the 1948 base period and the 1960 forecast are shown in figure 4. The 1960 gradient shape is similar to the shape for trial one except for a slight decrease in the growth allocated to the inner rings, which caused a lessening in the bowing effect and in the slope of the gradient in the intermediate areas. Forecast results are discussed after the other models have been described.

Accessibility Model

The generalized form of the accessibility model is

$$G_i = G_t \frac{A_i^a V_i}{\sum_i A_i^a V_i} \qquad (1)$$

Where,

G_i = Forecast growth for zone i.
G_t = Total regional growth $= \sum_i G_i$.
A_i = Accessibility index for zone i.
V_i = Vacant available land in zone i.
a = Empirically determined exponent.

The computation of the accessibility index traditionally is

$$A_i = \sum_j \frac{E_j}{(T_{ij})^b} \qquad (2)$$

Where

E_j = Measure of activity in zone j.
(T_{ij}) = Traveltime from zone i to zone j.
b = An empirically determined exponent.

However, friction factors developed by Voorhees in the gravity model calibration were used in the computation of accessibility for the research reported here, as

$$A_i = \sum_j E_j (F_{ij}) \qquad (3)$$

(F_{ij}) is the friction of time separation of zones (T_{ij}) minutes apart. The (F_{ij}) values are approximately proportional to the actual number of trips (T_{ij}) minutes long per trip end in each pair of zones (T_{ij}) minutes apart. In practice the computation of (F_{ij}) is complicated by a desire to have the (F_{ij}) results form a smooth monotonic relation to (T_{ij}) while maintaining approximate equality between the resultant mean trip length.

By using the foregoing explanation of the model, only the parameter for the empirically derived exponent, a, had to be determined. Two methods could be used: (1) Results from studies in other cities could be used to forecast 1960 zonal growth for an independent test of the model or (2) by calibration to actual changes in dwelling units from 1948 to 1960. Both methods were used to establish this parameter and the results are discussed after the description of other models. A value of 2 was assumed for parameter a in the first method; Hansen (*2*) had established that $a = 2.7$ was the optimum value for this parameter in the Washington, D.C., area. The assumption that accessibility would have less influence in shaping growth in a smaller city was substantiated by the results obtained when fitting values for a. Methods used in fitting a to the 1948–60 data are described at the end of this article.

Regression

Expressing the dependent variable of the multiple regression formulation as a function of the 1948–60 growth rather than as a function of the absolute amount of cumulative development at a single point in time was considered desirable. This function of growth was used to maintain comparability with the dependent variables of the other models and to conform to the standard practice of transportation planning models (*8*). As pointed out by the Traffic Research Corporation, greater accuracy is expected for relatively short-range forecasts when predicting increments of growth (*9*).

By using change in dwelling units or some function thereof as the dependent variable, an independent forecast to check against the 1960 data could not be obtained. The equation parameters had to be estimated from the full 1948–60 data files. Therefore, in contrast to the other four methods, accuracy results for the regression model of land use forecasts are shown only for a fitted model and not a forecast. Dwelling unit data for a third point in time would be required to test the forecasting reliability of the calibrated regression equation.

The regression approach differs from the other models used in the study in two ways: Many independent variables as opposed to one or two may be incorporated; and variables are related to growth only in linearly weighted combination, but they may be transformed prior to regression. The latter restraint is imposed by the standard regression program. The BIMD 34 stepwise multiple regression program developed by the UCLA Bio Medical Center for the IBM 7090/7094 was used in this work. Nonlinear regression equations may be developed, but different normal equations must be solved and standard regression programs cannot be used.

Equations developed

Many equations were developed for testing different hypotheses regarding the functional relation between variables. Forty-four independent variables plus selected nonlinear transformations were examined. These included measures of zone size and amount of land in different uses; accessibility to employment; time and distance to HVC; zonal employment, total and by major type; 1948 densities; vacant available land; zoning protection; land value; and proportion of total land and developed land in each major use. Four definitions of the dependent variable tested were the increase in dwelling units (D.U.), the log D.U., the D.U. per unit of available land (D.U./L.), and the log D.U./L.

Logarithimic transformations were used to test hypotheses for exponential relations, as expressed in the accessibility model. The growth-per-unit-of-available-land transformations were used to remove measures of zone size from the equations, thereby avoiding distorted relations caused by peculiarities of area definitions.

The equation accepted after all trials was:

$$Y = -2.3+0.061X_1+0.00066X_2+1.1X_3 -0.11X_4-0.0073X_5 \qquad (4)$$

Where,

Y = Logarithm of growth in dwelling units per unit of vacant land for 1948–60.
X_1 = Zoning protection in 1948.
X_2 = Percentage of total land area in residential use in 1948.
X_3 = Logarithm of accessibility to employment in 1960.
X_4 = Dwelling unit density in 1948.
X_5 = Percent of total land in industrial use in 1948.

The coefficient of correlation is 0.61. The t and β (standardized regression coefficient) values are shown in table 1 for each of the independent variables in the equation. Regression coefficients are significantly different from zero, with 95 percent of confidence. The transformed accessibility variable has the greatest β value and exhibits the most influence upon the estimate of the dependent variable. The percentage of industrial urban land has the lowest β value and, therefore, contributes least to the total equation estimate.

Table 1.—Relative significance and explanatory power of variables in regression equation

	t	β
Log accessibility to employment in 1960	4.30	0.321
Zoning Code, 1948	2.89	.213
Percentage of total land in residential use, 1948	2.70	.187
Dwelling unit density, 1948	3.28	.177
Percentage of urban land in industrial use, 1948	2.98	.159

Dwelling unit density

The contribution of each independent variable to the equation is discussed in the following paragraphs. The zoning code was a rating from 0 to 9, where a high rate indicated zoning control close to a single family residential area, and a low rate indicated marginal to no zoning control. The positive relation reflects the positive environmental influence of strict residential zoning policy. The positive contribution of percentage of residential development is a measure of residential clustering. Slow growth or decline in the residential density in old city areas coupled with the rapid increase in fringe and newly settled areas accounts for the negative coefficient for dwelling unit density. The negative contribution of percentage of industrial land use reflects an environmental restraint on new residential development. The positive contribution of accessibility is self-explanatory.

Logarithmic and intensity unit estimation introduced operational difficulties. The estimating equation could neither accept negative rates for the dependent variable nor estimate zonal decline. Therefore zones that suffered dwelling unit decline over the calibration period were considered to have shown no change. Several zones had a dwelling unit growth but had no vacant land available in 1948. Without adjustment the growth intensity value is infinite. Therefore, large arbitrary growth intensity rates were substituted. Unlike other models, there is no built-in provision to assure that the accumulated zonal estimates obtained from the regression equation solution will equal the actual total regional growth. All regression estimates had to be factored up to sum to the actual regional growth.

Two Intervening Opportunities Models

The two opportunities models tested were based on different initial assumptions and had dissimilar mathematical forms. Nevertheless, both models can be reduced to a simple general hypothesis that the probability of acceptance for development of a suitable residential opportunity (a unit of available capacity) is a monotonically decreasing function of the number of intervening opportunities. These opportunities were ranked by time from the HVC. Improvement in these models could perhaps be made by allocating increments of growth from all major centers of employment in proportion to the amount of employment in each center. This would make the test of the intervening opportunities models more comparable to the accessibility model procedure.

Stouffer model

The Stouffer model is defined as:

$$g_p = \frac{ko_p}{o} \qquad (5)$$

Where,

g_p = Dwelling units forecast to be located in a particular area, p.
o_p = Opportunities in interval p.
o = Opportunities from central point through interval p.
k = Constant of proportionality to assure that the total number of dwellings located equals the actual total growth.

As stated, the Stouffer model can be applied without assuming any values for parameters. However, the operation requires that the study area be structured into a number of discrete geographic units which are ranked from a central distribution point, the HVC in the research discussed here. One method of totaling areas, according to Strodtbeck (*10*), is to delineate a small number of rings containing approximately equal numbers of opportunities. For the initial application of the Stouffer model to the allocation of residential growth, the Greensboro study area was divided into 10 rings; each ring was composed of a number of zones and an approximately equal number of opportunities. Zones were assigned to rings according to their ranking in time from the HVC. By direct substitution in the formula, it was possible to determine g_p, the forecast number of dwellings in ring p. The ring forecasts were then proportioned among the constituent zones on the basis of opportunities.

To explain the fitting of the Stouffer equation to 1948–60 data the equation must be converted into its continuous differential form, as follows:

$$d(G_p) = \frac{kd(o)}{o} \qquad (6)$$

By integrating:

$$G_p = k \cdot lno + C \qquad (7)$$

Where,

G_p = Dwellings allocated to all opportunities from the central point up to and including opportunity interval p.
$d(G_p)$ = Dwellings allocated to opportunity interval p.
o = As defined in equation 5.
$d(o)$ = Opportunities in interval p.
C = Constant of integration.

This equation plots as a straight line of slope k, where the ordinate, total allocated dwellings, is in linear form; and where the abscissa, total accumulated opportunities, is a logarithmic scale. As a test of the appropriateness of the Stouffer model in describing the spatial distribution of residential growth in Greensboro, the actual accumulated zonal dwelling unit growth 1948–60 was plotted against accumulated 1948 opportunities. Zones were ranked by traveltime to the HVC. The straight line plot of figure 5 tends to confirm the validity of the Stouffer model. Because a single straight line could not be adequately fitted to the points, two distinct straight lines were necessary. The two lines were hand fitted, the 1960 growth estimates were made to the individual zones from the straight lines, and the error was computed. These results and those computed from the initial, noncalibrated test of the Stouffer model are discussed later with the results of the other four models.

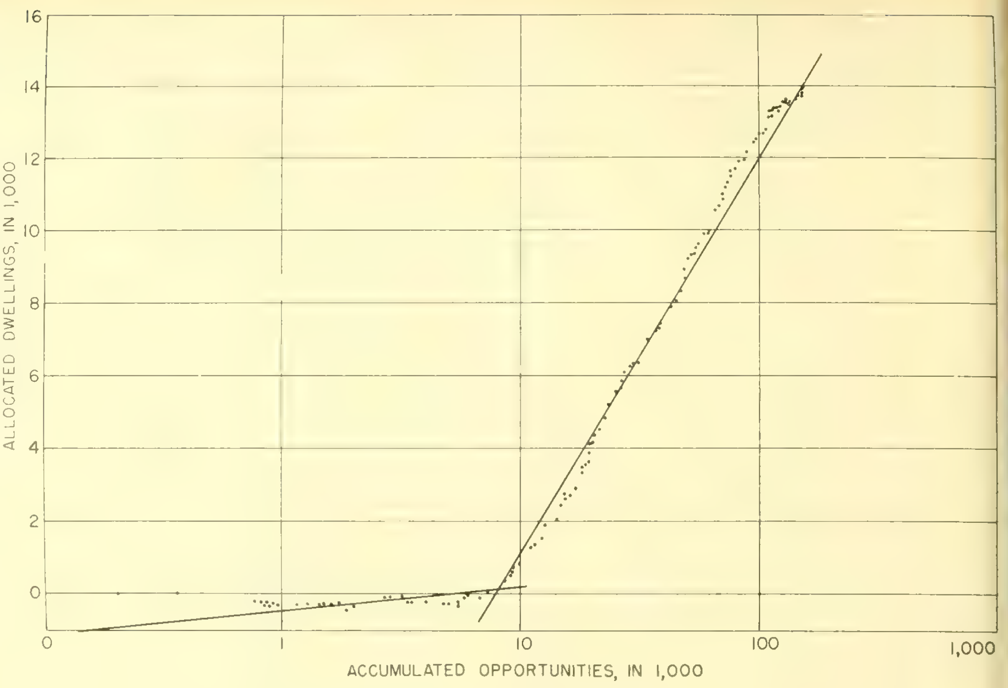

Figure 5.—Test of Stouffer model.

Schneider model

As applied to the distribution of residential growth, the Schneider model is:

$$d(G_p) = g_t[(e^{-\ell o}) - e^{-\ell(o+o_p)}] \qquad (8)$$

Where,

G_p = Locations in opportunity interval from the central point up to interval p.
$d(G_p)$ = Locations in opportunity level p.
g_t = Total growth to be allocated.
ℓ = Model parameter expressing probability of an opportunity being accepted for location.
o = Opportunities from the central point up to interval p.
o_p = Opportunities in interval p.

The parameter ℓ must be stipulated to apply to this model. The first trial of the model for a 1960 forecast was completed without the 1948–60 data. The value of ℓ was estimated from the assumption that the actual dwelling unit increase within the study boundaries was 99 percent of the aggregate Greensboro oriented growth. The resultant ℓ was 12.76×10^{-6}. This theoretical model is based on a distribution to an unbounded area. Application to a finite area requires specification of the number of accepted opportunities outside the boundary or the percentage of accepted opportunities up to the boundary.

For an explanation of the fitting of the Schneider model to 1948–60 data, the equation can be restated after integration as

$$G_p = g_t[1 - e^{-\ell o}] \qquad (9)$$

Subtracting g_t from both sides and rearranging

$$g_t - G_p = g_t e^{-\ell o} \qquad (10)$$

or

$$\ln(g_t - G_p) = \ln g_t - \ell o \qquad (11)$$

This relation plots as a straight line where the ordinate $(g_t - G_p)$, is in logarithmic scale and the abscissa, total accumulated opportunities from the central point (o), is in linear scale. The slope is ℓ and the intercept g_t.

The Schneider model effectively replicated the spatial distribution of residential growth in Greensboro. Therefore, plotting the actual quantity $(g_t - G_p)$ against accumulated opportunities (o) in semilogarithmic forms yielded linear relations, as shown in figure 6.

Straight line segments

As with the Stouffer model, the Greensboro data plot as two distinct straight line segments rather than one, as required by the initial model formulation. The zones comprising the transition area between the two straight line segments (fig. 6) are the same ones as those at the juncture of the two line segments for the Stouffer model (fig. 5). The fitted line slopes can be loosely compared to the short and long trip ℓ's, which are standard practice in applying the Schneider model as a trip distribution model. The slope for the central city line segment is 1.707×10^{-6}, and for the outer suburban area it is 10.9×10^{-6}.

As shown in figures 5 and 6, the distribution of residential growth in Greensboro from 1948 to 1960 did not adequately conform to results of either of the intervening opportunities models for the complete range of opportunities. However, the data plot as two straight lines, as shown in both figures. The transition points in the vicinity of the intersection of the fitted straight lines in both figures are for the same data points and represent the same zones. Although a detailed examination of these zones has not been attempted, it seems that they approximate a transition ring in Greensboro that separates the inner city, marginal growth area from the suburban, rapid expansion area. This band encircles the HVC at a radius of 1½ to 2 miles. In 1948 Greensboro had a leveling off in the percent saturation gradient at 3½ to 4½ miles from the HVC. The area circumscribed by this transition band was probably characteristic of areas in most cities that are old, perhaps show signs of blight, and have little available residential capacity.

The inner area straight line slopes drawn to the two plots are both very close to the horizontal. However, steep slopes represent the suburban areas. Viewing the opportunity surface from the HVC according to either of the two plots, a hypothetical locator assesses himself a greater penalty in passing up suburban opportunities than inner city opportunities. The significantly lower slopes on the plots indicate that the inner city opportunities are a less desirable subset of the total; therefore a lower probability exists for acceptance of individual opportunities. Perhaps those making location choices from the inner city opportunity subset are responsive more to the individual living qualities of the opportunities other than to their accessibility. The inner city opportunities may be considered to have homogeneous access whereas the suburban subset of opportunities are differentiated by access.

Stability

Of a purely forecasting interest is the question of the stability of the linear relations shown in figures 5 and 6—do the slopes remain more or less constant over time and how does the transition area behave in relation to the total opportunity surface. A speculation may be made that the straight line relation fitted to the data will hold and that the diffusion in residential location in the past is merely a reflection of the diffusion in the opportunity surface. A physical dispersion outwards caused by the filling in of less distant areas would occur rather than an alteration in the location function. But, it is possible that through time the slopes of the plots may be flattening out, which is symptomatic of a society less restrained by the impedance of travel. Clearly, answers to such speculations are required before the applicability of the fitted lines to forecasting to a future time point can be estimated.

Interpretation of Results

The results obtained by testing the five residential land use forecasting techniques and the actual change in residential structure of the Greensboro area are presented in the following paragraphs. The outcome of the trial forecasts are diagnosed by comparisons with actual growth and some conclusions have been made.

Naive model

The single accuracy measure that was calculated for all trial forecasts was the sum of the squares of the dwelling unit forecasting error. The accuracy measures were computed for the sectors, rings, districts, and zones. A sixth forecast was made using the naive assumption of equal growth for all zones. The computed error sum of squares, which will be referred to as the naive model, is (n-1) times the variance in actual zonal residential growth. The naive model served as a bench mark in evaluating the results of the five techniques.

Time and distance

The computed error sum of squares for all of the forecasts and calibrations at each level of aggregation is shown in table 2. For complete comparisons, the results of zone level forecasts for each of the models, except the DSGM model, have been collected to districts and rings defined by both time and distance from the HVC. Trial one of the DSGM was based on analysis at the district level defined by distance from the HVC. Trial two of the DSGM was based on analysis at the district level defined by time to HVC.

The error measurements shown in table 2 are an index that can be used to compare results for the same level. However, comparisons between data in the different columns are meaningless as different numbers of areas and different variances from mean growth rates are involved at different geographic levels. Comparisons between forecast techniques as well as between geographic analysis levels can be made from the ratio of all errors to errors obtained in forecasts with the naive model, as shown in table 3.

Zone level results

Forecasting results with all five models were poor at the zone level. Sometimes results from the naive model, assuming equal growth for all zones, exceeded the accuracy of other forecasts. Discouraging zone level results obtained with the DSGM indicated poor choice of criteria by the authors in distributing growth from districts to zones. Accurate use of the DSGM model requires historical data that were not available and intimate familiarity with the area, which the authors lacked. The DSGM model cannot be blamed for the results obtained in the research reported here.

Table 2.—Error sum of squares for all trials[1]

Model	Zone	Districts		Rings		Sector
		Distance ring	Time ring	Distance	Time	
DSGM:						
Trial 1	2.33	6.97		8.36		9.69
Trial 2	2.41		4.43		4.07	3.02
Accessibility:						
Forecast	1.80	4.16	2.84	3.25	2.33	4.58
Fitted	1.79	3.98	2.76	2.18	1.99	4.46
Regression:						
Fitted	1.85	4.71	3.14	5.16	2.84	3.71
Stouffer:						
Forecast	2.21	6.45	4.22	5.57	3.48	11.25
Fitted	1.91	4.72	3.07	2.42	1.46	8.84
Schneider						
Forecast	2.07	6.16	4.13	4.10	3.38	13.92
Fitted	1.95	4.65	3.08	1.91	1.65	10.18
Naive	2.20	7.66	5.22	20.64	10.54	16.18

[1] All results have been multiplied by 10 [illegible]

Table 3.—Ratio of all errors to naive model error

Model	Zone	Districts		Rings		Sector
		Distance ring	Time ring	Distance	Time	
DSGM:						
Trial 1	1.06	0.91		0.41		[illegible]
Trial 2	1.10		0.85		0.39	.19
Accessibility:						
Forecast	.82	.54	.54	.16	.22	.28
Fitted	.81	.52	.53	.11	.19	.28
Regression:						
Fitted	.84	.62	.60	.25	.27	.23
Stouffer:						
Forecast	1.01	.84	.81	.27	.33	.70
Fitted	.87	.62	.59	.12	.14	.54
Schneider:						
Forecast	.94	.80	.79	.20	.32	.86
Fitted	.89	.61	.59	.09	.15	.63
Naive	1.0	1.0	1.0	1.0	1.0	1.0

A substantial amount of the error at the fine detailed zone level can be attributed to inaccuracies in data, such as assumptions made in certain estimates, incompatibility of merged files, definition differences between time periods, etc. However, other factors also contributed to inaccurate forecasts. The average zone contained 109 dwelling units in 1948 and the number of dwelling units had increased to only 165 by 1960. These figures were too small to obtain a reliable prediction with any model. Obviously, differences in land use between zones at this level were primarily the result of random variations that could not be reflected by the models. The districts represented a more reasonable level of detail at which to compare the accuracy of forecasts. For comparison with transportation study practices, the average district—defined by distance rings—used in the study could be expected to have about 8,000 person trip ends in 1948—about 660 dwelling units with 3.2 persons per dwelling and four trip ends produced per person.

Accessibility model accuracy

The relative accuracy of the accessibility model forecasts at different levels in relation to the size of the rates being forecast are shown in table 4. The root mean square

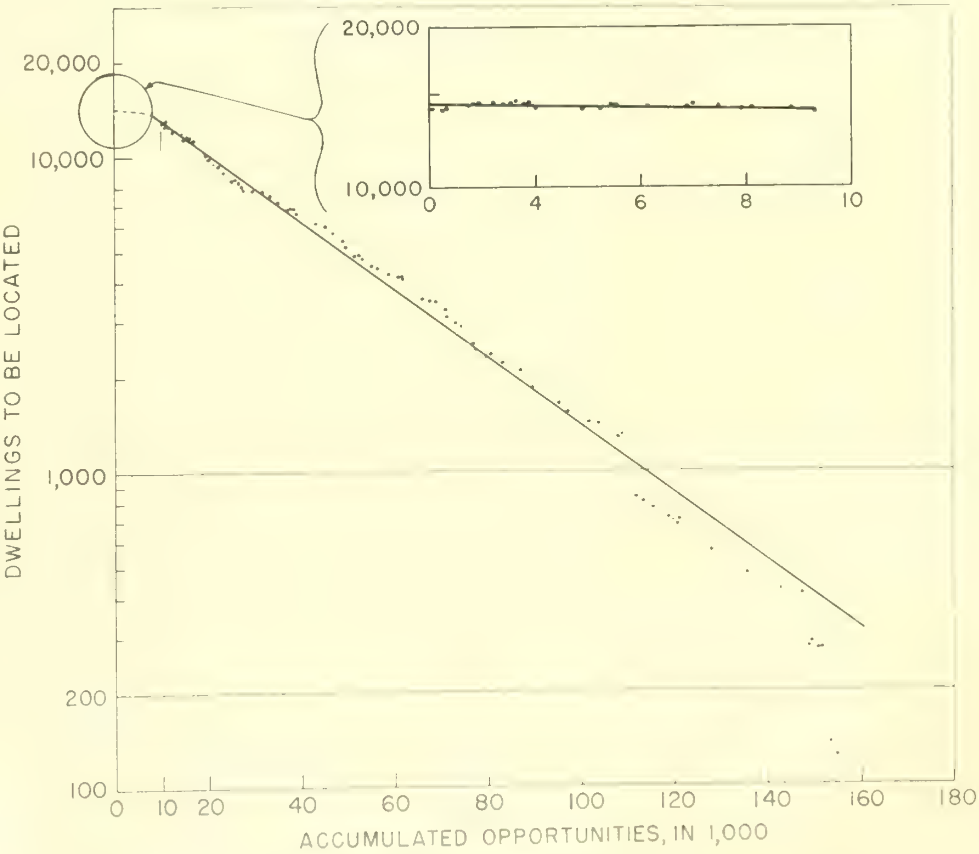

Figure 6.—Test of Schneider model.

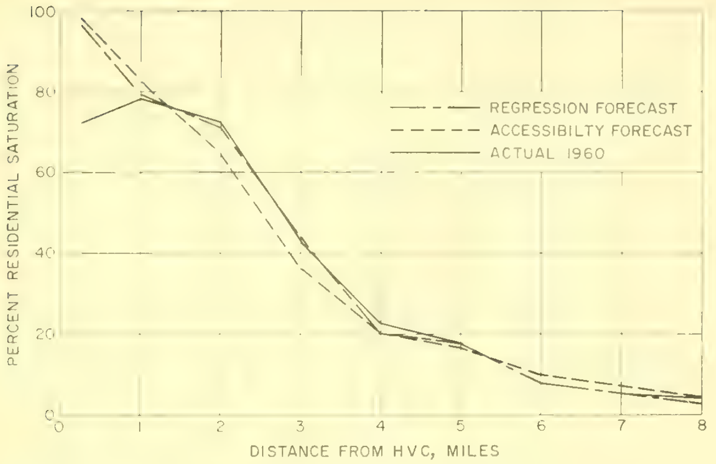

Figure 7.—Comparison of 1960 and 1948 dwelling unit densities by distance bands.

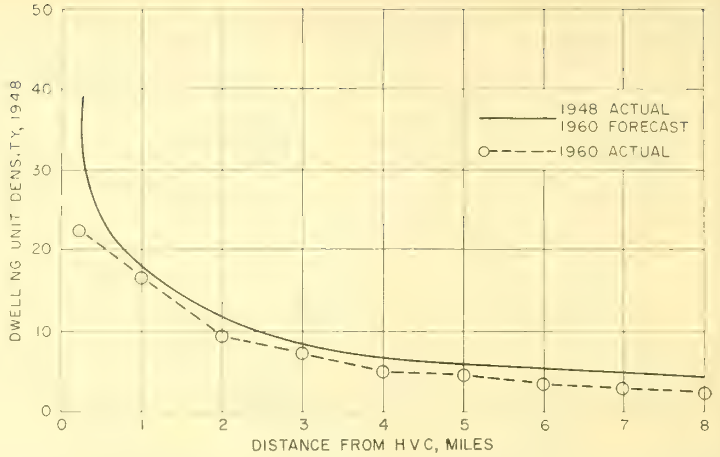

Figure 9.—Regression model and accessibility model forecasts compared with actual 1960 percentage of residential saturation.

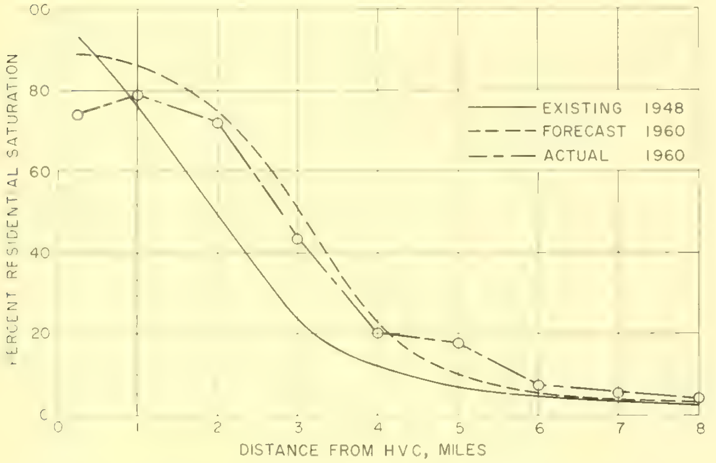

Figure 8.—Density-saturation gradient model forecast compared with actual 1960 and 1948 percentage of residential saturation.

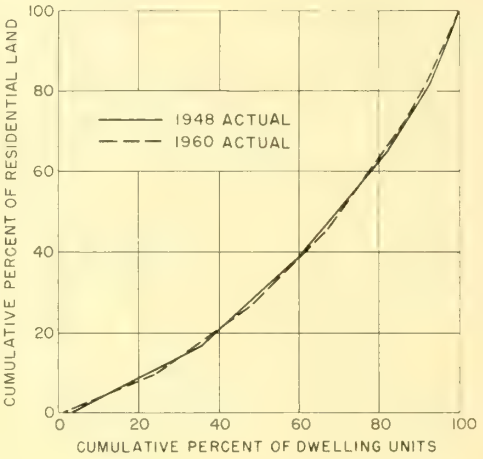

Figure 10.—Cumulative percentage of residential land and dwelling units for 1948 and 1960 from the core area.

Table 1.—RMSE of accessibility model related to actual growth

	Zones	District, distance bands	Rings, distance bands
RMSE, $\sqrt{\frac{S.S}{n}}$	85	381	600
Dwelling units:			
Average per area, 1960	165	1,006	4,580
Average growth per area, 1948–1960	56	342	1,560
Total number of areas	249	41	9

Table 5.—Results of three versions of linear regression on transformed accessibility model

	Model 1	Model 2	Model 3
Accessibility exponent, b	3.52	1.63	2.29
Log a	−8.0	−3.2	−4.9
Vacant land exponent, c	1.51	1	1
Sums of squares of errors[1]	2.21	1.89	1.78

[1] Results have been multiplied by 10^{-6}.

error (RMSE) is used as the measure of error as it can be compared to the size of the forecasts: About two-thirds of the errors being within 1 RMSE. The RMSE was roughly half the average number of dwelling units per zone for 1960, and about a third the average number of dwelling units per district for 1960. These accuracies must be considered in relation to the overall growth rate of 52 percent. The ratios of the RMSE's to the 1960 results could be expected to be nearly cut in half if the overall growth rate were half as large. As shown in table 3, at most levels the accessibility model provided substantially better results than other unfitted models but the results from the fitted Stouffer and Schneider models were comparable to those obtained from the fitted accessibility model. Addition of several explanatory variables did not improve the accuracy of the linear regression model.

Results at the sector level are important because of their implication for forecasting radial corridor movements. The intervening opportunities models produced comparatively poor results here, perhaps because these models were not made sensitive to the distribution of employment as were the accessibility regression models.

For trial one of the DSGM, relative growth was assumed by sectors in proportion to available capacity—a weak assumption when judged by the error of trial two. Residential character is important for attracting additional growth at all levels: Between sectors, as demonstrated by comparison of the two DSGM trials, and at the zone level, as demonstrated by the statistical significance of residential land in the regression analysis.

Growth patterns

All 1960 density forecasts were based on the assumption that development in any zone would occur at the density indicated by a smooth line drawn through the 1948 density points with reference to distance or time from the HVC. The actual 1960 density-distance gradient is compared with that for 1948 in figure 7. There was a uniform decrease in density at all distances, except for the core area where the decrease was substantial. This difference undoubtedly accounts for some error in the core area forecasts that required estimates of 1960 density for the DSGM and the intervening opportunities models.

The actual 1960 and 1948 percentage of saturation gradients are given in figure 8,

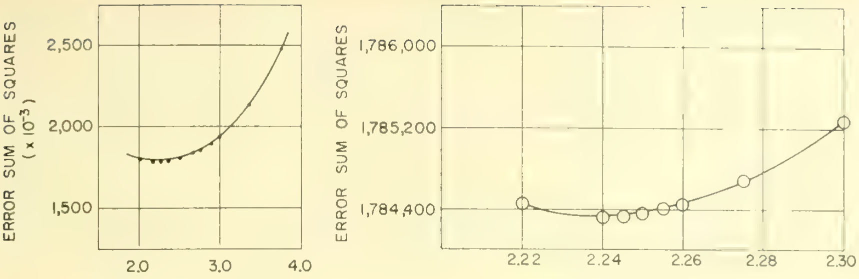

Figure 11.—Accessibility model sum of squares of error for a range of exponents.

along with the forecast curve used for trial one of the density-saturation gradient method. The actual 1960 curve is not as smooth as the 1948 curve as the plot represents the percentage of 1948 capacity rather than 1960 capacity. The most significant errors in the forecast seem to have been the result of a large decline in the density at the core and the growth in relatively remote parts of the area as rings 5 and 6 indicate. However, the general shape of the forecast curve is appropriate.

Figure 9 shows the density-distance results obtained from the accessibility and regression models. Agreement with the actual 1960 gradient is very good, except for the obvious inability of these models as used in the research to predict dwelling unit decreases in the core area.

Plots shown in figure 10 reflect the changes in the residential density structure from 1948 to 1960. The cumulative percentage of total regional dwellings was plotted against the cumulative percentage of total residential land area on the HVC ring level basis. Percentages proceed outwards from the core ring. Plots for the actual conditions in both 1948 and 1960 are shown. If smooth curves were drawn the slope at any point would represent the inverse marginal dwelling unit density. A diagonal line drawn on figure 10 would represent uniform residential density for the entire study area. The bowing of each of the curves below the diagonal indicates the decline in density outwards from the HVC. If there were a decline in the inner area densities and an increase in the dwelling unit density in the outer rings, the total area would be approaching a state of uniform density, and the curve would shift toward the diagonal. But, if the difference between inner and outer area densities was to increase substantially, there would be a shifting of the plot down and to the right. The overall increase in residential density of 52 percent from 1948 to 1960 indicates a rather minute change in the density structure of the study area.

Although the two plots in figure 10 appear to coincide almost exactly, they do not indicate an absence of change in the geographic distribution of dwelling units from 1948 to 1960. That is, inasmuch as the majority of residential growth occurred in the suburban rings, the dwelling units in the inner rings in 1960 represent a smaller part of the total area dwelling units than in 1948 and they occupy a smaller part of the available residential land.

It is unknown whether similar plots for other urban areas have the consistency of the Greensboro area plots. If this is determined to be so, such plots could be helpful in making residential land use forecasts.

Calibration of Accessibility Model

Two procedures used to estimate the best exponent of accessibility were linear regression on transformed variables and an iterative, nonlinear least squares fit of the untransformed dependent variable. Both are reported here as the differences in results may be of interest.

Linear regression on transformed variables

Three transformed equations of the accessibility model also were tested; they are

$$\log G_i = \log a + b \log V_i + c \log A_i$$

or, in nonlogarithmic form

$$G_i = a(V_i)^b (A_i)^c \qquad (12)$$

$$\log \frac{G_i}{V_i} = \log a + b \log A_i$$

or, in nonlogarithmic form

$$G_i = a(V_i)(A_i)^b \qquad (13)$$

$$\log G_i - \log V_i = \log a + b \log A_i \qquad (14)$$

The nonlogarithmic form for equation 14 is the same as for equation 12. The nonlogarithmic forms of (12) and (13) are essentially the same as the standard form of the accessibility model. They would be identical if they contained the condition that

$$a = \frac{G_T}{\sum V_i (A_i)^b} \qquad (15)$$

As a standard regression program was used, this condition might be violated and equation estimates must be factored to sum to actual total growth. This holds for all three of the transformed versions of the model.

Equation 12 also expresses vacant land as a power function in contrast to its linear form in the standard formula. The basic problem, however, is that the least squares criterion is different for each equation (the minimization of unexplained variance in the dependent variable) as the dependent variable is different for each. None is the correct criterion. The log transformed equation tends to produce a bias toward better fits for the smaller values of the untransformed dependent variable.

The results of the three versions of the accessibility model are summarized in table 5. Because of the fairly large variation in the accessibility exponent, as well as in the error term, the authors are skeptical of regression on transformed dependent variables.

Nonlinear least squares fit of exponent

A routine was programed to iterate the true least squares solution for the standard accessibility model

$$G_i = G_T \frac{(A_i)(V_i)^b}{\sum_i A_i (V_i)^b}$$

Results are shown in a plot of the sums of squares of error in reference to a range of exponents in figure 11. A smooth curve with a minimum sum of squares at $b = 2.24$ is shown. The right part of figure 11 is a blowup of the critical area of the plot shown on the left. These results can be compared with the b value of 2.7 reported by Hansen (*2*) for Washington, D.C.

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State Highway Patrol Functions and Financing

BY THE
OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by [1, 2] **EDWARD A. GLADSTONE and**
THOMAS W. COOPER, Financial Economists,
Highway Statistics Division

The primary purpose of the study reported in this article was to investigate the method used by each State for law enforcement agency financing. The organizational structure, statutory authority, scope of responsibility, and method of financing that is used in each of the States was examined in order to establish a basis for comparing their methods of financing. The agency structures and methods of financing vary among the States; but the problems arising from increased highway mileage and traffic are generally shared throughout the States. Increased highway mileage and traffic have forced law enforcement agencies to take on additional functions that in turn generate an increased need for manpower and revenue.

The Bureau of Public Roads has determined that the most reasonable and usable method for financing the varied police functions in all the States would be to support traffic-related functions from highway-user revenues and nontraffic-related functions from general funds. It is believed that this plan would provide adequate finances for the additional manpower required to meet the growing needs of traffic and traffic safety. Some States have adopted the plan indicated by the Bureau of Public Roads. In other States, however, application of this plan is complicated by the problem of determining which functions are traffic-related and which are nontraffic-related. Another difficulty arises in determining the type of highway-user tax to be used to support traffic functions. In deciding these issues, the States have developed individual plans and legislation to provide for their law enforcement agencies.

The problem of law enforcement agency financing is not resolved, however. The need for additional manpower and finances continues to increase; the emphasis on traffic safety expressed by all State officials indicates that the problems of traffic law enforcement and its financing remain major issues.

Introduction

THIS ARTICLE is a report on a study of State highway law enforcement agency financing. A structural examination was made of law enforcement agencies including agency organization, status within the State government organization, present statutory powers, and functions performed, in 49 States (Hawaii was not included). In all States, traffic patrol functions include law enforcement and public service. But, the specific structures, functions, and methods of financing the traffic patrol agencies are different. Comparative analyses of the agencies were made to measure the relative adequacy of the patrols in the individual States.

Since 1921, the Bureau of Public Roads has annually compiled and published highway finance statistics; since 1925, it has identified receipts and expenditures relative to highway policing. These statistics have been continually improved, expanded, and refined so that today law enforcement agency finances can be thoroughly documented. However, costs of law enforcement cannot as yet be assigned to precise functions. In some States the costs of routine traffic supervision cannot be separated from costs of driver and safety education, vehicle size and weight enforcement, and other functions; some State agencies are responsible for enforcing criminal laws as well as traffic laws, and the costs of each may not be identifiable. Nevertheless, findings from this study should be useful to police organizations, public authorities, and legislative bodies in measuring the needs and accomplishments of the patrol agencies, as well as suggesting equitable financing plans, that appropriately reflect the functions of the agencies.

Background

Most of the Governors of the 47 State legislatures meeting in 1965, voiced concern for the safety of the motoring public, and the importance of the State police organization enforcement of traffic and safety laws. In State after State, the Governor's message emphasized the need for additional highway patrol troops to curb highway accidents and fatalities. Governors, legislative committees, and safety agencies, in at least 30 States requested increased patrol strength. Collectively, 21 States requested that nearly 3,800 troopers be added to the patrol strength during the next 1 to 4 years. As of mid-1965, the States have requested a 23 percent increase in patrol strength. Although requests probably will not receive full legislative approval in many States, they are an indication of State concern for combating accidents and fatalities.

Governor George Romney of Michigan pointed to some significant statistics in his Special Message on Traffic Safety to the 1965 legislature. He labeled the 2,125 deaths, 145,000 injuries, and 285,000 reported accidents of 1964 the "Michigan Massacre"; it caused an estimated economic cost of at least $380 million to Michigan and its citizens, or more than $1 million a day. Statistics adapted from the Governor's report show that the rate of increase of traffic mishaps between 1962 and 1964 in Michigan was considerably larger than the increase in vehicles or travel. Traffic deaths increased 35 percent from 1962 to 1964; traffic injury, 34 percent; property damage accidents, 22 percent; economic losses, 33 percent; vehicle registration, 10 percent; licensed drivers, 5 percent; vehicle-miles of travel, 13 percent.

Governor Romney's Special Commission on Traffic Safety concluded that legislation, manpower, and financial and public support were inadequate to keep pace with the mounting problem. This Commission typifies the concern of State governments in the mid-1960's for highway safety and the necessity for adequate law enforcement practices and policies for reducing accidents.

Conclusions

State traffic law enforcement agencies have gained many new functions over the years. But, the delegation of responsibility for additional functions, lack of manpower, and the method of financial support for the new functions have been problems.

These problems are exemplified with respect to toll roads. The toll road capacity to provide many services and aids to motorists has been publicized, but the scope and means for providing the services have not been resolved. At present the police agencies are not equipped with the manpower to provide motorist aids. Recruitment would not be difficult, but additional manpower costs cannot be covered in all States under their present method of financing. Information from the questionnaires received from all the States shows that police agencies would require a total of 56 percent increase in personnel by 1969; based on current costs, this increase would require revenues of more than $450 million in 1969.

The majority of the traffic law enforcement agencies are supported from highway-user

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[1] Presented at the 45th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1966.

[2] The assistance of the State law enforcement agencies, as members of the International Association of Chiefs of Police, especially William Franey, Director, and his staff of the Highway Safety Division, is acknowledged. The State law enforcement agencies responded to a questionnaire designed jointly by the authors and IACP staff members and circulated by IACP.

revenues either through direct allocations from highway-user taxes or indirectly through general fund appropriations. The support of traffic-related functions from highway-user revenues is recognized by the Bureau of Public Roads as a proper highway expenditure. Therefore, it would seem reasonable that highway-user revenues would support traffic-related functions and general funds would support nontraffic-related functions. Some States have already adopted this plan of financing.

Adequate financing of highway patrols is also related to safety. The increased mileage of divided highways is followed by increased traffic accidents and fatalities each year. Although there is no conclusive evidence that increased patrol strength or functions would reduce accidents, the requests by Governors and legislators for additional personnel indicate a belief that such a correlation exists.

Origins and Development

The law enforcement arm of the States began nearly 150 years ago with the first border patrols. Traditionally, the responsibility for law enforcement in our general structure of self-government has been concentrated at the local level. With few exceptions this practice, which dates back to early British institutions, shaped our law enforcement procedures until the 20th century. The first State law enforcement agency was the Texas Rangers, formed in 1835 to patrol the Mexican border. Arizona and New Mexico also formed State border patrols in the early 1900's, but they were soon abolished because of political involvement.

The first State police force or constabulary, as it was initially called, was formed in Massachusetts in 1865. This agency was created primarily to suppress commercialized vice, but it was granted general police powers throughout the State. In 1879, this agency was absorbed into the Massachusetts District Police, a State detective unit. Its new duties included investigating fires, enforcing fish laws, and inspecting boilers and buildings. This agency was absorbed into the Department of Public Safety in 1920.

In 1903 the Connecticut State Police was patterned after the Massachusetts District Police and was chiefly responsible for enforcing liquor and gambling laws. In 1905, the Pennsylvania State Constabulary was organized, and this organization marked the beginning of a new era in rural police administration. From the beginning, it operated as a mounted and uniformed body having a widely distributed system of troop headquarters and substations as a base of operations. This provided a continuous patrol throughout the rural areas. Broad administrative powers were granted the Superintendent of State Police, who was responsible only to the Governor. In 1917, New York organized a State police force.

When the Nation was comprised of widely separated municipalities, local law enforcement was adequate. But, as society became more mobile and population increased, a centralized mechanism became necessary if law and order were to prevail. In some areas sheriffs and constables became unable to cope with crime and were reluctant to enforce unpopular State laws; waste, mismanagement, and political influence caused the municipal police function to deteriorate. There was a need for uniformity of State law enforcement methods; and there was a lack of coordination of activities in an era of mobile crime.

Governor Arthur M. Hyde expressed the gravity of the situation in his message to the 52d Missouri General Assembly in 1923. He stated that, "Nowhere is there any effective agency for enforcement of law and maintenance of order except the National Guard. The State owes an inescapable duty to the public to preserve peace and order. . . . No law can be enforced without the cooperation of three officials—sheriff, prosecuting attorney, and court. When one of these three fails, anarchy results.

". . . No power exists whereby the State can send any of its officials into the county to assist in preserving peace and order. Unless the emergency is grave enough to warrant sending the National Guard, the State and the people are helpless.

". . . The best machinery for law enforcement by State authority yet devised is a State police force. A State constabulary is the remedy, so far as remedy exists in the powers of government against lawlessness."

State Police Forces

Police facilities were greatly improved during the 20-year period from 1919 to 1939 by the creation of State police forces in 46 States. Paralleling this action was the increased use of automobiles, which created the need for motor-vehicle and traffic law enforcement agencies. But, combining the functions of traffic law enforcement with criminal law enforcement caused much controversy. Local governments were reluctant to relinquish general police powers to a central agency. Labor unions were opposed to a strong central police force because of the alleged strike-breaking activities of the Keystone Police in Pennsylvania.

From the controversy two separate types of police departments evolved; one had broad police powers, the other had only highway law enforcement power. The distinction is expressed in a statement from *The Book of the States*, 1962–63 edition, page 437, published by the Council of State Governments:

". . . The State police enforce all laws, including traffic laws and regulations and their enforcement arms reach into every corner of the State. . . . The duties of most State highway patrols, important though they are, are restricted almost entirely to enforcement of traffic laws and regulations and to carrying out highway accident-prevention programs. . . ."

The separation of highway law enforcement and general criminal law enforcement remained a controversy. Local officials in many States opposed centralization of police functions and allowed the new highway agencies only limited power. Against this, it was argued that a general police force should be given sufficient authority to combat crime in general as highway patrolling played a [illegible] the apprehension of criminals. In Texas and Pennsylvania parallel agencies were created to enforce criminal laws and motor-vehicle laws. However, the separate agencies were subsequently consolidated. In other States that have highway patrols, the local government retains general police power.

Initial traffic enforcement legislation in some States did not create a police organization, but merely authorized the appointment of individuals to patrol the highways. These patrolmen enforced traffic laws, motor-vehicle regulations, and sometimes all State laws. In time, the State legislatures became interested in general policing and many of the traffic patrol agencies of the 1920's and 1930's were reorganized as State Police. By 1941, 35 State Police agencies were authorized to enforce all State laws; 13 organizations had limited powers.

Legislative Restrictions

Broad police powers, however, were still subject to legislative restrictions. Enabling legislation in many States denied patrolmen jurisdiction in criminal investigation or authority to search and seize. Patrolmen were only permitted to take weapons from arrested persons; the case was then referred to duly authorized peace officers. The legislature also restricted law enforcement agencies through control of funds. The general fund and highway-user revenues that supported the law enforcement agencies were tightly controlled by the legislature.

Legislative control of highway patrol activities through control of revenue was exercised in Missouri. Initially the Highway patrol in that State had only highway and motor-vehicle law enforcement powers. Later, broad police powers were granted and exercised. The constitutionality of these broad powers was challenged by the State's Attorney. The Missouri Constitution provided that highway moneys should be used for highway purposes only. Because the Highway patrol was supported entirely by these moneys, general police powers for the Highway patrol were considered unconstitutional. A subsequent legislature authorized that the State general fund pay 10 percent of the Highway patrol budget, thereby limiting the general policing activities of the Missouri Highway Patrol to 10 percent of its activities.

Agencies also are restricted to highway authority on the premise that traffic enforcement has little need for detective or central investigation powers. However, the Highway patrol is expected to perform certain duties of a nonhighway nature such as the investigation of crimes originating on the highway; arrest of criminals committing crimes in the presence of a patrolman; apprehension of criminals using highways for escape; and aid to local peace officers or the Governor on request. But, these nontraffic functions are generally confined to rural areas.

Changing needs made periodic reexaminations of existing statutes necessary. As the volume of traffic increased, traffic enforcement agencies were forced to expand numerically and to assume broader regulatory functions. By 1941, 25 police departments were reorganized as independent organizations. The remaining 23 State agencies were subsidiary units of such departments as public safety, highway, motor-vehicles, law enforcement, and public works and revenue.

Highway Police Organization

Today's highway police organization has expanded in form, organization, and purpose, and the trend continues toward the granting of broad police powers. Broad police powers have been granted to State police agencies in 43 of the 49 States covered in the study reported in this article; the police powers of only 6 agencies are limited to highway-related functions. However, the trend toward the creation of independent agencies has been reversed; only 19 of the 49 States have independent agencies. This includes New York and Rhode Island, where the State police are divisions of the executive departments, but have independent status; and West Virginia where the department of public safety and the State police are the same organization. In the other 30 States, police agencies are subdivisions of a department of public safety, highway department, motor vehicle department, highway safety department, department

Table 1.—Statutory provisions governing creation and financial support of law enforcement agencies

State	Primary agency	Citation creating agencies	Source of funds	Citation apportioning or appropriating operating funds
Alabama	Department of Public Safety.	Title 36; Sec. 58(54).	Motor vehicle fees.	Title 36; Sec. 58(61), 53.
Alaska	Department of Public Safety.	Sec. 44.41.010 (AS).	General fund.	Ch. 167, Laws of 1961.
Arizona [1]	Highway Patrol Division.	Sec. 28.231–A (ARS).	Highway-user revenues.	Sec. 28–231D.
Arkansas	Department of State Police.	Sec. 42–402 (1947 Stat.).	Driver license fees and general funds.	Title 76–309.3(c)(a).
California [1]	Department of Highway Patrol.	Sec. 13975, Gov. (WCAC).	Motor vehicle fees.	Ch. 3, Sec. 42.271(b).
Colorado	State Highway Patrol.	Sec. 120–10–1 (CRS, 1953).	Highway-user revenues.	Sec. 120–10–23.
Connecticut	State Police Department.	Title 29; Ch. 529, Sec. 29–4 (GSA).	Highway-user revenues and general funds.	Title 14; Ch. 246, Sec. 14–156.
Delaware [1]	State Police Division.	Ch. 83, Sec. 8301, Title 11 (DCA).	General fund.	Title II; Sec. 8303.
Florida	Department of Public Safety.	Sec. 321.01 (FSA).	General fund.	Sec. 321.09.
Georgia	Department of Public Safety.	Ch. 92A–105 (GCA).	Driver license fees through general fund.	Ch. 92A–120.
Hawaii [2]				
Idaho	Department of Law Enforcement.	Sec. 19.4801 (IC).	Highway-user revenues.	Sec. 19.4811.
Illinois	Department of Public Safety.	Ch. 121, Sec. 307.1 Supp. (IAS).	Motor vehicle fees.	Statute 127, Sec. 144.3.
Indiana	State Police Department.	Ch. 8, Sec. 47–846 (BIS).	Highway-user revenues and general funds.	Title 36–2817.
Iowa	Department of Public Safety.	Sec. 80.1 (ICA).	General fund.	Sec. 80.8; Ch. 1, Sec. 51, Laws of 1963.
Kansas	State Highway Patrol.	Ch 74, Art. 20a01 (GS–1949).	Highway-user revenues.	Ch. 68–416a.
Kentucky	Department of Public Safety.	Ch. 17, Sec. 010 (KRS).	Highway-user revenues and general funds.	Ch. 3, Laws of 1962.
Louisiana	Department of Public Safety.	Sec. 40:1301 (WLRS).	Driver license fees and general funds.	Ch. 2, Sec. 32.412 and 426; Ch. 75, Laws of 1962.
Maine	State Police Department.	Ch. 15, Sec. 1 (RS–1954).	Highway-user revenues and general funds.	Ch. 23, Sec. 133.
Maryland	Department of State Police.	Art. 88B, Sec. 3 (ACM–1957).	Motor vehicle revenues and general funds.	Art. 66.5, Sec. 341.
Massachusetts	Department of Public Safety.	Ch. 22, Sec. 1 (GLA).	Highway-user revenues and general funds.	Ch. 22, Sec. 9B.
Michigan	State Police Department.	Ch. 24, Sec. 4.432 (SA).	General fund.	Ch. 193, Sec. 1, Laws of 1961.
Minnesota [1]	Highway Patrol.	Sec. 161.47 (MSA).	Highway-user revenues.	Sec. 161.47, Subd. 5.
Mississippi	Department of Public Safety.	Title 30; Ch. 2, Sec. 8078 (MC–1942).	Motor vehicle fees through general fund.	Title 30; Ch. 2, Sec. 8120.5.
Missouri	State Highway Patrol.	Title 5; Sec. 43.020 (VAS).	Highway-user revenues and general funds.	Title 5; Sec. 43.100 and 43.180.
Montana	State Highway Patrol.	Title 31–101 (RSM–1947).	Driver license fees and general funds.	Title 31–105 and 31–210.
Nebraska [1]	Law Enforcement and Safety Patrol.	Title 60.431 (RSN–1943).	General fund.	Ch. 330, Laws of 1963.
Nevada [1]	Highway Patrol Division.	Ch. 481.130 (NRS).	Highway-user revenues and general funds.[3]	Ch. 481.083 and 48:150.
New Hampshire	Department of Safety.	Ch. 106.A:1 (NHRSA–1955).	Highway-user revenues and general funds.	Ch. 6.12.
New Jersey	Department of Law and Public Safety.	Ch. 52.17B–3 (NJSA).	General fund.	Ch. 52.9H–2.
New Mexico	State Police Department.	Ch. 39, Article 2, Sec. 2, (NMS–1953).	Driver license fees through general fund.	Ch. 39.2–22.
New York	Division of State Police.	Executive Law, Sec. 210.	General fund.	Ch. 147, Laws of 1964.
North Carolina [1]	State Highway Patrol.	Ch. 20–1 (GSNC).	Highway-user revenues.	Ch. 20–194.
North Dakota	State Highway Patrol.	Ch. 39–03–02 (NDRC–1943).	Motor vehicle fees and general funds.	Ch. 39–03–08 and –16.
Ohio	Department of Highway Safety.	Ch. 5503.01 (ORCS).	Motor fuel tax and motor vehicle fees.	Ch. 4501.6.
Oklahoma	Department of Public Safety.	Title 47; Ch. 2, Sec. 103 (OSA).	Driver license fees and general funds.	Title 47, Sec. 6–101, 14–116, 22.2; Ch. 296, Laws of 1963.
Oregon	State Police Department.	Title 18; Sec. 181.020 (ORS).	Highway-user revenues through general fund.	Ch. 181.040.
Pennsylvania	State Police Department.	Title 71; Sec. 65 (P–P).	Highway-user revenues through general fund.	Title 72; Sec. 3563a.
Rhode Island	Division of State Police.	Title 42; Ch. 28, Sec. 2 (GLRI) 1956.	General fund appropriation.	Sec. 42–28–32.
South Carolina [1]	Law Enforcement Division.	Sec. 46–851 (CLSC–1962).	Highway-user revenues.	Sec. 33–287.
South Dakota [1]	Motor Patrol.	Sec. 44.06.03 (SDC–1939).	Highway-user revenues.	Sec. 44.06.06.
Tennessee	Department of Safety.	Sec. 4.320 (TCA).	Motor vehicle fees through general fund.	Sec. 4.711.
Texas	Department of Public Safety.	Art. 4413 (VCS).	Highway-user revenues.	Arts. 6687b(15), 6701d(141), 4413(26).
Utah	Department of Public Safety.	Title 41–13–1; (VAC–1953).	Motor fuel tax and motor vehicle fees.	Title 41–13–4.
Vermont	Department of Public Safety.	Ch. 111, Sec. 1811 (VSA).	Highway-user revenues and general funds.	Ch. 252, Part III, Laws of 1961.
Virginia	Department of State Police.	Title 52; Ch. 1, Sec. 52–1 (CV–1950).	Motor vehicle fees.	Title 46.1; Sec. 167(c,2).
Washington	State Patrol Department.	Ch. 43, Sec. 43.010 (RCW).	Driver license fees and motor-vehicle fees.	Ch. 46.68.30 and .40.
West Virginia	Department of Public Safety.	Sec. 1237 (WVC–1961).	Learner's permit fees and general funds.[4]	Sec. 1721(213); Ch. 12, account 570, Laws of 1963.
Wisconsin [1]	Enforcement Division.	Ch. 110.01 (WWSA).	Motor vehicle fees.	Ch. 20.560(73).
W[illegible]	Highway Patrol.	Title 31; Ch. 2, Sec. 31.7 (WS–1957).	Highway-user revenues.	Sec. 31–11.

[1] [illegible]gh these are secondary agencies, they represent the primary State agency responsible for law enforcement. In these States, the parent department is concerned primarily with activities other than police work.

[2] [illegible] State police organization; each island has its own police department.

[3] [illegible] activities of the highway patrol, when authorized by the Governor, are supported by the general fund.

[4] [illegible] permit fees are deposited in the general fund to the credit of the State police.

of law enforcement, department of safety, department of law and public safety, or highway transportation agency. The agency having the law enforcement function in each State is listed in table 1.

Since 1941 changes have occurred in the organization of police agencies. The number of subordinate organizations has only increased by 5, but 12 subordinate agencies and 8 independent agencies have reorganized. Nine of the reorganized agencies were placed under different departments; three became independent. Of the eight independent agencies that reorganized, six became units within the highway department; and one became a unit within the department of motor vehicles.

Agency Organization

Although the organizations of many police agencies have comparable status within the State government, their internal structures are not alike; most of the structural differences can be explained by the differences in functions. For example, Pennsylvania and Maryland operate as State police organizations; New Jersey and Texas, as departments of public safety; and Missouri and Washington, as highway patrols.

Broad Police Powers

The Pennsylvania State Police organization has six bureaus and a field division of troops. The Technical and Staff Services Bureaus are under the chief of staff and provide administrative services for the organization; the Training and Staff Inspection Bureaus are under the commissioner; and the Detective and Traffic Bureaus are under the deputy commissioner and form a line operation with the field division troops. The field division consists of 15 substations each containing crime, staff, and traffic units. The internal structure is a recent development; the primary functions of the police have been set apart from supporting services to ensure the most efficient operation.

The Maryland State Police organization is divided into a headquarters office of superintendent, executive officer, and adjutant, who supervise eight divisions. An intelligence unit on the same organizational level reports to the superintendent in the headquarters unit. The eight divisions are: Training-personnel; Investigation; Quartermaster; Communications; Accident Records; Medical; Finance; and Operations.

The New Jersey State Police is one of seven divisions of the Department of Law and Public Safety. The police organization is divided into three major categories of responsibility. Five police commands form the field operations and report to the superintendent. A deputy superintendent supervises administrative activities; an executive officer directs operations communications activities; and an investigation officer directs criminal investigation and identification activities.

The Texas Department of Public Safety administered by the Public Safety Commission through a director and assistant director has a headquarters unit composed of four major divisions, each headed by a chief; seven special service sections; and the Rangers. Also, six regional commands are organized as counterparts of the headquarters structure and have direction and identification as: Supervisor, Communications; Technicians, Crime Laboratory; Adjutant, Office Services; Sergeant, Safety Education; Captain, Driver License; Captain, Highway Patrol (may be more than one district); Captain, License and Weight; Captain, Motor Vehicle Inspection. Each regional commander reports to the director.

The Pennsylvania, Maryland, New Jersey, and Texas agencies are responsible for the enforcement of both criminal and traffic law. But the Missouri and Washington organizations are structured primarily for traffic activities.

Traffic law enforcement

In Missouri, the State Highway Patrol headquarters unit is divided into six divisions that report to an assistant superintendent. Nine patrol troops in a field operations unit are under the direction of two field supervisors, who also report to the assistant superintendent.

The Washington State Highway Patrol has many divisions. A chief directs the Investigative and the Finance and Budget Divisions; an assistant chief supervises six divisions; an administrative officer directs seven divisions; a staff inspector supervises field forces in seven districts, a Program and Planning Division, and a General Maintenance Division.

The Police Function

A tendency seems to exist to organize the police into functional units for criminal law enforcement, traffic law enforcement, training, communications, and so on. Traffic supervision and crime repression are the major functions; the former is carried out in every State police agency, the latter in most agencies. Related functions are organized as divisions within the agency. The agencies are staffed by both uniformed and civilian personnel. The uniformed or sworn personnel, designated as peace officers, troopers, patrolmen, etc., have authority to apprehend and arrest. The civilian personnel perform clerical tasks and in many States operate communication networks.

Traffic supervision is the primary function in each agency and requires the major part of each State patrolman's time. Traffic supervision includes the enforcement of traffic laws, supervision and direction of traffic, and the investigation of accidents. Within the framework of enforcing traffic laws, the patrolman must (1) observe all vehicle use and users, roadway and vehicle conditions, and deter would-be traffic law violators; (2) detect pertinent defects in individual behavior, vehicle equipment, or highway condition; (3) initiate appropriate action to prevent such defects from causing accidents or delays, remedy the defects, or discourage repetition of dangerous or prohibited acts; (4) investigate complaints of traffic law violations; (5) record and report all activity; and (6) assist the courts during adjudication of traffic violations.

In supervising and directing traffic the patrolman (1) provides information that will help people to reach their destinations and to comply with traffic laws and regulations; (2) indicates to drivers what is desired and expected of them, particularly when and how to move in congested areas; (3) takes emergency action to direct flow of traffic when signals or controls prove to be inadequate; and (4) provides assistance by escort, as authorized.

Support functions

When investigating accidents, the patrolman is required to (1) take action to prevent aggravation of the damage and injury; (2) and obtain information on the circumstances of the accident, determine the specific violation of law, and record and report all information.

Several essential supporting functions, performed at a technical or supervisory level, implement the basic traffic functions. They include (1) maintaining records; (2) compiling data on needed corrective action; (3) preparing special studies or reports; (4) coordinating plans and activities with official agencies and support groups; (5) supplying laboratory and technical aids; and providing incidental services to motorists. Cooperative functions are also performed by the police in conjunction with other agencies. Safety education, driver examination, driver record maintenance, vehicle inspection, vehicle weighing, equipment regulation, bicycle inspection and regulation, and suspension and revocation notification are considered cooperative functions.

Many States have the automobile theft investigation and recovery function located within the activities of the police agency. It is usually classified under the criminal investigation rather than traffic function. Agencies with broad police powers have criminal law enforcement functions that are generally assigned to a detective bureau, indentification bureau, and a crime laboratory. There are other miscellaneous functions that are assigned to only a few police agencies. They cover such activities as fire prevention and investigation; firearms regulation; livestock inspection, theft, and patrol; boat registration; liquor control; fish and game law enforcement; building and boiler inspection; underwater recovery; inspecting migrant workers' homes, boarding homes, and nursing homes; and civil defense. Supporting activities that are essential to the efficient operation of the organization include personnel, finance, quartermaster, planning, maintenance, special services, internal inspection, training, and communication.

Traffic function time

The amount of time devoted to traffic- and nontraffic-related functions varies among agencies. A composite distribution, summarized in table 2, was made based on percentages supplied from a questionnaire sent to each State police agency. Of primary interest was the time spent on basic traffic supervision. In agencies that are subordinate units of a department of public safety, about 47 percent of the total activity of both uniformed and civilian personnel was devoted to

Table 2.—Distribution of organization functions—uniformed and nonuniformed

	Traffic functions	Traffic-related functions	Other functions
ALL PERSONNEL			
Subordinate agencies	*Percent*	*Percent*	*Percent*
Departments of public safety	46.8	42.4	10.8
Highway, motor vehicle, and other	64.0	30.2	5.8
Independent agencies			
State police	47.4	26.7	25.9
Highway patrol	51.0	46.2	2.8
UNIFORMED PERSONNEL			
Subordinate agencies:			
Departments of public safety	68.0	22.0	10.0
Highway, motor vehicle, and other	75.0	20.7	4.3
Independent agencies			
State police	58.5	20.3	21.2
Highway patrol	72.1	24.8	3.1

Table 3.—Sworn uniformed and civilian personnel for 1940 and 1964 [1]

State	Sworn uniformed personnel		Civilian personnel		Total	
	1940	1964	1940	1964	1940	1964
Alabama	134	561	27	204	161	765
Alaska		101		46		147
Arizona	41	319	10	66	51	385
Arkansas	61	218	7	60	68	278
California	719	2,795	197	907	916	3,702
Colorado	116	332	16	141	132	473
Connecticut	225	502	85	201	310	703
Delaware	92	221	15	53	107	274
Florida	60	614	4	596	64	1,210
Georgia	168	420	12	326	180	746
Hawaii [2]						
Idaho	40	140	46	27	86	167
Illinois	350	1,145	127	525	477	1,670
Indiana	221	651	85	232	306	883
Iowa	128	285		82	128	367
Kansas	67	223	5	80	72	303
Kentucky	113	457	10	198	123	655
Louisiana	439	554	26	491	465	1,045
Maine	109	244	11	47	120	291
Maryland	104	733	34	249	138	982
Massachusetts	323	614	105	164	428	778
Michigan	443	1,170	131	305	574	1,475
Minnesota	125	377	13	86	138	463
Mississippi	85	378	18	118	103	496
Missouri	175	505	48	440	223	945
Montana	71	143	9	66	80	209
Nebraska	67	225	3	45	70	270
Nevada	11	50	3	16	14	66
New Hampshire	52	126	12	17	64	143
New Jersey	319	996	70	242	389	1,238
New Mexico	42	204	4	66	46	270
New York	895	2,464	34	282	929	2,746
North Carolina	188	648	41	185	229	833
North Dakota	13	80	5	6	18	86
Ohio	200	855	60	409	260	1,264
Oklahoma	125	348	18	253	143	601
Oregon	168	535	17	91	185	626
Pennsylvania	1,516	2,015	141	312	1,657	2,327
Rhode Island	71	119	10	21	81	140
South Carolina	154	460	25	2	179	462
South Dakota	17	88		36	17	124
Tennessee	100	518	32	200	132	718
Texas	340	1,398	98	1,014	438	2,412
Utah	50	152	6	41	56	193
Vermont	37	111	26	53	63	164
Virginia	178	745	42	304	220	1,049
Washington	167	369	50	380	217	749
West Virginia	218	279	32	85	250	364
Wisconsin	45	222	2	187	47	409
Wyoming	15	75	1	11	16	86
TOTAL	9,397	26,784	1,773	9,968	11,170	36,752

[1] Source of the 1940 data, *State and Provincial Police*, by David Geeting Monroe, p. 9.
[2] Hawaii has no State police organization; each island has its own police department.

traffic supervision; in agencies that are subordinate units of highway and motor vehicle departments about 64 percent was spent; in independent State police agencies 47 percent was spent; and in independent highway patrols 51 percent was spent. Where agency activity was confined to sworn uniformed personnel, averages were higher. Traffic supervision occupied 68 percent of the total activity in department of public safety agencies compared to 75 percent in subordinate police agencies, 59 percent in State police agencies, and 72 percent in highway patrols.

Information from the questionnaire showed an average workweek of 48 hours for uniformed personnel and 40 hours for civilians. The data also were used to determine the time spent on agency functions by highway patrolmen. Criminal investigation and its supporting functions were not considered. The patrolman spends 40 hours a week in the performance of basic traffic functions, including traffic surveillance, accident investigation, auto theft and recovery, and court appearances; 1 hour in compiling records and statistics and in police laboratory work; 4 hours in driver licensing, truck weighing, motor-vehicle and school bus inspection, and safety education; and 3 hours in communications, personnel, training, and special services required within the internal structure of the organization.

Manpower

Each State police agency was also asked to report its personnel strength for three periods of time. Civilian and sworn uniformed personnel were considered separately. As of July 1, 1959, there were 22,864 sworn uniformed personnel and 7,981 civilians or a total force of 30,845 persons engaged in State police activities. By July 1, 1964, the number had increased to 26,784 sworn uniformed personnel and 9,968 civilians, a total force of 36,752. The sworn uniformed personnel increased 3,920 or 17 percent and the civilian strength 1,987 or approximately 25 percent. The total strength of the police agencies increased 5,907 or 19 percent.

Estimates by each agency show that 57,444 persons including 44,210 sworn uniformed personnel and 13,234 civilians will be needed to provide adequate traffic supervision and perform the other agency functions by July 1, 1969. This is an increase of 17,417 troopers or 65 percent, 3,582 civilians or 33 percent, and a total increase of 20,699 or 56 percent. The growth of police agencies over the 25 years from 1940 to 1964 is shown in table 3. The increase of 25,582 in police manpower during this 25-year period is compared to an expected increase of 20,699 during the 5-year period 1964–69.

Questionnaire responses showed that 29 State agencies were responsible for patrolling State highways within incorporated municipalities. Of those agencies, five patrolled the highways upon request of local governments; five confined urban patrol to the Interstate System; four patrolled small cities and towns; four patrolled where no local police force was maintained; and the remaining State patrols gave no explanation for patrolling these highways. Most State patrol agencies devoted less than 5 percent of their total activity to patrolling municipal areas; six States devoted 5 to 10 percent to incorporated areas.

County roads in 41 States are patrolled by the State agency. In addition, county or township patrols operate in 39 States. Out of 3,054 counties, 773 road patrols and 320 township road patrols are operated. Eight States have road patrols in each county; 10 States have no local county road patrols. Separate patrols for the free sections of the Interstate System exist in only five States. However, as more mileage is opened to traffic and motorist needs increase, the need for these patrols should also increase.

Statutory Authority

The statutory provision for the creation and financial support of law enforcement agencies in each State is listed in table 1.

Good roads amendments

Presently 29 State constitutions have good roads amendments that earmark highway-user taxes for highway purposes. A model amendment published in *Good Roads Amendments* by the National Highway Users Conference, Inc. in 1965 proposed that: "No monies derived from fees, excises, or license taxes relating to registration, operation, or use of vehicles on the public highways, or to fuels used for the propulsion of such vehicles, shall be expended for other than cost of administering laws under which such monies are derived, statutory refunds and adjustments provided therein, payment of highway obligations, cost of construction, reconstruction, maintenance and repair of public highways and bridges, and expense of State enforcement of traffic laws." Such amendments are designed to prevent future legislatures and special interest groups from diverting road-user taxes to other than highway purposes.

Constitutions in 29 States include good roads amendments. Funds are specified for motor-vehicle and traffic law enforcement in 13 State constitutions. These States allocate highway-user funds directly, with the exception of Alabama and Oregon, where tax revenues are appropriated to the enforcement agencies from the State general funds.

Six State amendments provide that highway funds support traffic supervision or traffic safety. In this group Idaho, New Hampshire, South Dakota, and Colorado support law enforcement agencies by direct allocations. In Pennsylvania, moneys are appropriated from the special motor license fund, to the general fund, and then reappropriated to the policing agency. Though receipts pass through the general fund they are specifically designated for the highway patrol agency. In Iowa, the good roads amendment does not provide for the support of the highway patrol from highway revenue. However, operator and chauffeur license fees deposited in the general fund provide substantial support for the highway patrol.

In 10 States, the amendments do not include traffic enforcement as a highway cost. Within this group Kansas and Minnesota have amendments that earmark highway funds for highway purposes. Kansas legislation provides highway fund support for a traffic patrol; Minnesota allows the highway patrol to be placed under any agency as long as patrol functions remain the same and patrolling costs can be covered by highway revenues. In Florida and Georgia, road-use moneys apply only to motor-fuel tax receipts, and in Florida only to two-sevenths of the receipts. The Florida Department of Public Safety is supported by the general fund; but, driver license fees deposited in the general fund are considered support for the highway patrol agency. The Georgia Department of Public Safety is also supported by driver license fees that are deposited in the general fund. However, in Georgia the department has first claim to these funds. In Louisiana driver license fees are credited directly to the law enforcement agency.

Prior to April 1, 1963, Michigan's amendment restricted motor-fuel and vehicle registration revenues to highway construction, maintenance, and administration. State police functions were not supported from these revenues. On April 1, 1963, the State constitution was changed to provide highway revenues for highway purposes, as defined by law. Governor Romney subsequently recommended the use of highway funds for the freeway patrol. The Michigan amendment does not specifically cover fees on operators' licenses; the fees are deposited in the general fund where they are drawn upon to support, in part, the State police.

General funds

The traffic enforcement agencies in Montana, North Dakota, and West Virginia are financed principally from the State general fund. Their amendments do not provide for traffic law enforcement as a highway cost element. In Montana and North Dakota, revenue from operators' license fees is not subject to the purview of the amendment; these revenues are deposited in the State general fund, from which appropriations are made for enforcement agencies. Nevada's good roads amendment includes administration as well as construction and maintenance as authorized highway expenditures. The legislature determined that Motor Vehicle Department costs should be considered administrative costs. Consequently, the Nevada State Highway Patrol, as a division of the Motor Vehicle Department, is financed entirely from highway funds. Generally, in States where constitutional amendments do not provide highway moneys for law enforcement, there is little the legislatures can do in assigning such moneys to cover highway police activities.

Although 21 States have no consitutional good roads amendments, in practice, most apply road-user tax revenues exclusively to highway purposes. Five of these States operate on a one-fund basis whereby all State revenues are deposited in a general fund from which highway appropriations are paid. Therefore, road-user revenues in these States support policing and other highway activities to the extent that revenues equal appropriations for highway and related purposes. In 11 of the States, road-user revenues are allocated directly for support of the traffic enforcement agencies. In the remaining five States, the agencies are supported almost entirely from general fund appropriations. In four of these five States, operator license and road-user revenues placed in the general fund partially or entirely equal the amount expended by police agencies.

In summary, the costs of enforcing State traffic and motor-vehicle laws is clearly recognized as a highway expense by 21 of the 29 States having good roads amendments; by 11 of the remaining 21 States; and by many national organizations associated with roads and motor-vehicles. This interpretation is shared by the Federal Government, as explained in the following paragraph:

The Federal Interpretation

The Bureau of Public Roads has for many years considered highway policing an essential and important highway activity, and it has ruled that expenditures for that purpose are fully consistent with the policy statement in the Hayden-Cartweight Act, 48 Stat. 993 (1934). The act states that it is unfair and unjust to tax motor-vehicle transportation unless the proceeds of such taxation are applied for construction, improvement, and maintenance of highways and administrative expenses in connection therewith. The terms of the ruling were defined by the Administrator of the Bureau of Public Roads through the General Counsel. Administrative expenses, were interpreted to include not only the administration of highway construction and maintenance but also the enforcement of motor-vehicle and traffic laws.

Policing Toll Roads

Traffic patrol functions on public toll roads can be effectively analyzed and measured because such roads consist of a known mileage and carry known volumes and types of traffic. The special services provided on toll roads, however, are not necessarily applicable to tax supported freeways such as the Interstate System. The major toll roads are generally patrolled by personnel from State police agencies; however 15 toll roads have permanently assigned detachments. Established patrol units are supported entirely from toll revenues paid by the road users. The initial cost of permanent toll road patrols is usually appropriated from the parent law enforcement agency, subject to reimbursement by the toll road authority.

Available data showed that more than $11 million was spent for policing on 19 major toll roads during 1964. These funds were almost entirely devoted to traffic law enforcement; criminal investigation represented only a minor expense. The traffic law enforcement function included maintenance of an orderly flow of traffic as well as motorist assists. Motorist assists, necessary because of widely separated interchanges and service centers on toll roads, is provided by State police and service crews commissioned by toll road authorities.

Although maintenance and emergency service vehicles are available, the patrol is usually the first to provide aid. The patrolman will manage minor emergencies himself, but obtains assistance for serious situations. Some aids given motorists are: gas; tire changes; service calls; mechanical aid; fire extinguishing; information; parking and U-turn permits; checks on sleeping drivers; person, property, and message relays; special escort services; removal of objects from road; collection of unpaid tolls; checks on abandoned vehicles; removal of hitchhikers;

Table 4.—Statistics on highway patrols, 1964

Toll facility	Uniformed personnel	Road mileage	Expenditures	Annual vehicle-miles of travel	Motorist assists	Assists per trooper	Assists per million vehicle-miles of travel
	Number	*Miles*	*Thousands*	*Millions of miles*	*Number*	*Number*	*Number*
Indiana Turnpike	46	157	$556	612	22,018	4,786	36
Kansas Turnpike	23	237	303	341	14,855	6,459	44
New Jersey Turnpike	104	134	1,371	1,753	63,298	6,086	36
Garden State Parkway	98	175	966	1,552	33,586	3,427	22
New York Thruway	211	561	2,580	3,006	[1] 39,151	1,855	13
Ohio Turnpike	90	241	1,029	1,147	18,736	2,082	16
Dallas-Fort Worth Turnpike	13	30	137	197	13,097	10,074	66
Richmond-Petersburg Turnpike	19	35	193	213	8,523	4,486	40
West Virginia Turnpike	16	86	171	115	7,393	4,621	64
TOTAL	620	1,656	7,306	8,936	220,657	--	--

[1] Number of motorists with disabled vehicles aided by the Thruway patrol; an additional 46,101 motorists were assisted by Thruway emergency service crews.

Table 5.—Activity statistics on some toll road highway patrols, 1964

Toll facility	Miles patrolled (thousands)	Motorist assists	Traffic arrests	Warnings issued	Assists per thousand miles of patrol	Arrests per thousand miles of patrol	Warnings per thousand miles of patrol
Kansas Turnpike	1,371	14,855	2,139	1,614	11	2	1
New Jersey Turnpike	3,349	63,298	33,695	22,639	19	10	7
Garden State Parkway	3,400	33,586	15,856	20,770	10	5	6
New York Thruway	8,346	39,151	57,838	31,299	5	7	4
TOTAL	16,466	150,890	109,528	76,322			...

traffic direction; first aid; oversize vehicle fee collection.

Data is limited for comparing services on toll and toll-free highways. New Jersey, however, does furnish such information. A comparison of New Jersey State Police operations on toll (New Jersey Turnpike and Garden State Parkway) and nontoll roads during 1964 shows that in terms of the miles traveled on patrol, the incidence of vehicle arrests was higher on free roads and the number of motorist assists was considerably lower. A review indicates that 52 percent of the total functions of the patrols on toll roads in New Jersey were devoted to motorist assists in 1964, as compared to less than 1 percent for the other patrols. In contrast, arrests and traffic investigations accounted for only 32 percent of the patrol function on toll roads and 49 percent on nontoll roads.

Comparative data for patrol functions are not available for all major toll roads; however, comparisons can be made from information on specific toll roads as shown in tables 4 and 5. Table 4 contains information on personnel, road mileage, patrol expenditures, vehicle-miles of travel, and motorist assists by road patrols for nine of the Nation's major toll roads in 1964. Patrol expenditures averaged about $12,000 per man. Assists averaged 3,559 per trooper or in terms of travel, 25 motorists required assistance for every 1 million miles of travel.

A breakdown of motorist assists was obtained for a few toll facilities. It is estimated that 39 percent of the motorist assists involved mechanical failure; 20 percent involved tire failures; and 14 percent involved vehicles that ran out of fuel. The remaining 27 percent covered the other services performed by toll road patrols for the motoring public, such as message relays, special permits, special escorts, and first aid.

Table 5 charts three types of activity performed by toll road patrols in terms of the mileage patrolled on four roads. In addition to motorist assists, averages were computed for traffic arrests and warnings. Totaled, for every thousand miles of patrol, the trooper averaged nine motorist assists, seven traffic arrests, and five warnings.

Motorist Assists

If motorist assists are necessary on nontoll freeways, the question arises as to whether such assistance should be given by the State police or another highway agency. Governor Romney of Michigan stated, in a special legislative message on traffic safety, January 16, 1964, that, ". . . freeways must be patrolled for the supervision, protection, and assistance of our motorists." He referred to the fact that fewer traffic violations occur on freeways, although traffic supervision and police protection is still required in order to maintain an orderly and safe flow of traffic. Governor Romney pointed out that ". . . assistance to motorists, while incidental to the performance of regular police duties, is closely related to the basic police function of public protection."

Table 6.—Historical summary of expenditures for traffic law enforcement and related activities [1]

[*Millions of dollars*]

Years	Traffic law enforcement: Toll roads only	Traffic law enforcement: All roads	Safety education	Size and weight enforcement	Total
1946 [2]			----		41.3
1947 [2]		--	--	---	45.9
1948 [2]	--	-----	-	----	63.3
1949		67.8	1.3	1.1	70.2
1950	--	74.7	1.5	1.4	77.6
1951	0.2	86.2	2.6	2.4	91.2
1952	.6	89.2	3.8	2.5	95.5
1953	1.2	105.2	4.3	3.1	112.6
1954	1.8	115.7	4.7	3.2	123.6
1955	3.4	124.4	6.3	4.8	135.5
1956	4.7	139.0	7.4	5.2	151.6
1957	6.2	162.1	9.3	6.9	178.3
1958	7.5	183.3	10.0	9.0	202.3
1959	7.8	192.3	14.4	9.0	215.7
1960	8.6	208.9	15.9	10.4	235.2
1961	10.0	223.6	18.8	11.0	253.4
1962	10.5	235.5	24.9	12.8	273.2
1963	10.8	246.8	33.3	14.1	294.2
1964	12.0	263.1	48.4	15.3	326.8

[1] Includes expenditures by State highway departments and other State agencies for traffic related activities.

[2] No breakdown available.

In July 1962, there were 100 State troopers assigned to patrol Michigan freeways. However, a ruling of the Michigan Civil Service Commission limited troopers to a 48-hour week and freeway patrolling was discontinued for lack of manpower. The scheduling of freeway patrols was shifted from headquarters to local post commands. The net result was a sharp reduction in freeway patrols. But, the need did not disappear. The State Highway Commissioner initiated freeway service for stranded motorists on a limited scale and the Michigan Department of State Highways planned to expand the aiding of motorists to include the entire 1,000-mile freeway system at an estimated annual cost of $943,000. Governor Romney disapproved of the duplication of service by two organizations, and the attorney general subsequently ruled that motorist assistance by the Department of State Highways was illegal.

The Governor recommended an appropriation of $1,303,300 to finance a 130-trooper freeway patrol for fiscal year 1964–65. Aid to stranded motorists was to be a patrol duty. It was further recommended that support come from motor vehicle taxes rather than the general fund. At the time of this study, it was not known whether these recommendations became law.

Toll road authorities have a freer hand in authorizing services such as motorist assists and covering the costs incurred with user charges (tolls). But State highway departments and police departments, one of which must ultimately provide these aids, have the problem of balancing the costs of such services against the need for funds for other highway and traffic-related functions. Even where charges are levied for these aids it is not known whether revenue raised covers costs incurred.

Table 7.—Police agency disbursements for traffic and related functions, 1964

[Thousands of dollars]

State	Traffic supervision [1]	Safety education	Size and weight enforcement	Driver license examination	Motor vehicle inspection	Communications	Training	Administration and supporting services [2]	Total traffic-related disbursements	Nontraffic related activities	Total
Alabama	2,447	1,311		802					4,560	189	4,749
Alaska	657								657		657
Arizona	4,910								4,910		4,910
Arkansas	1,282	263		58					1,603	375	1,978
California	31,729	1,339	1,556			2,056	1,572	4,959	43,211		43,211
Colorado	3,854	5				328		427	4,614		4,614
Connecticut	3,525	161				344		439	4,469		4,469
Delaware	1,449	26							1,475		1,475
Florida	4,713			1,469		90		2,401	8,673		8,673
Georgia	3,851			958					4,809	1,366	6,175
Hawaii [3]	---										
Idaho	1,102	72	434						1,608		1,608
Illinois	12,448	244	684						13,376	612	13,988
Indiana	4,556	736	185					1,428	6,905		6,905
Iowa	3,022			417					3,439		3,439
Kansas	2,896							27	2,923		2,923
Kentucky	5,191	195		189					5,575		5,575
Louisiana	4,523	69	135	1,799	310			365	7,201		7,201
Maine	1,881								1,881		1,881
Maryland	5,036					355	181	2,772	8,344	359	8,703
Massachusetts	5,838								5,838		5,838
Michigan	7,096	286				369	87	1,142	8,980		8,980
Minnesota	5,214		41						5,255		5,255
Mississippi	3,179	121		309	93	332		787	4,821		4,821
Missouri	5,900	175	595	642				154	7,466		7,466
Montana	1,069			410				302	1,781		1,781
Nebraska	1,122	29				110	8	772	2,041		2,041
Nevada	637								637		637
New Hampshire	1,147	80				63		67	1,357		1,357
New Jersey	8,506		121						8,627		8,627
New Mexico	2,341	25		14		262	32	37	2,711		2,711
New York	19,345				387				19,732		19,732
North Carolina	6,117								6,117		6,117
North Dakota	709	15		37					761		761
Ohio	10,092	155		945				765	11,957		11,957
Oklahoma	2,941		236	372		246		47	3,842		3,842
Oregon	3,199					34		356	3,589		3,589
Pennsylvania	17,121								17,121		17,121
Rhode Island	788								788		788
South Carolina	3,314		87	373					3,774		3,774
South Dakota	996	45	116					24	1,181		1,181
Tennessee	3,452	90		480		77	6	787	4,892		4,892
Texas	6,597	194	750	2,853	668				11,062		11,062
Utah	1,663	23	375	373					2,434		2,434
Vermont	795	23	21			24		169	1,032		1,032
Virginia	6,077	167				835	250	1,058	8,387		8,387
Washington	3,663	270	654	808		1,108	79	1,105	7,687	694	8,381
West Virginia	1,588				97				1,685		1,685
Wisconsin	1,786		371			487	73	995	3,712		3,712
Wyoming	860	23							883		883
TOTAL	232,224	6,142	6,361	13,308	1,555	7,120	2,288	21,385	290,383	3,595	293,978

[1] Includes all costs of the agency when itemized expenditures were not available.
[2] Includes safety responsibility and pension funds.
[3] Not included in police study.

Police Financing

A study of the financing of State law enforcement agencies is complicated by the variety of organizational structures and the diveristy of functions performed. Moreover, many agencies keep financial accounts on an object basis, that is, salaries, supplies, equipment purchases, and so on, rather than on a functional or activity basis. Where the parent or primary department has a division structure and funds are accounted for on a division basis, the costs of the patrol usually can be identified. But, costs for traffic supervision, traffic-related functions, or general law enforcement are not easily distinguished. The related functions are not the same in all States, and allocations of funds to the functions often are not specifically recorded. Therefore, estimated assignments were made on the basis of personnel man-hours, activity reports, and the like.

Annual Expenditures

For the study reported here, law enforcement expenditures were derived from statistics published by the Bureau of Public Roads. The Bureau has compiled annual highway statistics since 1921 and has identified expenditures for law enforcement beginning in 1925. In that year California, Maine, and Pennsylvania reported expenditures for law enforcement activities totaling $924,000. By 1934, with 34 States reporting, expenditures totaled $8,800,000. In 1941, police expenditures reached $29,400,000 for 47 States; and by 1950, with all States reporting, expenditures were $77,600,000. The postwar boom in vehicle ownership, travel, and highway construction led to rapid increases in outlays for patrol operations; they doubled by 1956 to $151,600,000 and doubled again by 1964 to $326,800,000. As shown in table 6 expenditures for law enforcement have increased steadily since 1946.

Since 1949, the Public Roads statistics have identified traffic law enforcement, safety education, and vehicle size and weight enforcement as the major areas of law enforcement expenditure. However, because Public Roads classifies expenditures without regard to the expending agency, the expenditures given in table 7 are not necessarily limited to police agencies. Costs of certain services related to the general area of policing and safety are included in Public Roads statistical summaries even though they were not incurred by the police agencies. For example, inspection of motor vehicles may be done by patrolmen, employees of motor vehicle departments, or private garages; but the public cost is considered a law enforcement expense by the Bureau of Public Roads.

In using the Public Roads statistics, the method of accounting should be known. The only concern is with highway or traffic related functions, consequently, the financial reports published by Public Roads do not include any allocations and expenditures of funds for nontraffic-related functions except those financed by highway-user revenues. These expenditures are not recorded as a police cost, but as nonhighway expenditures of highway-user revenues.

As shown in table 1, the two major sources of revenue for police functions are highway-user taxes and general funds. Highway-user taxes are those levied on owners and operators of motor-vehicles for their use of the public highways. These taxes are primarily motor-fuel taxes, registration fees, operators' licenses, and other fees allied with the ownership and operation of motor-vehicles. Also included are fines and penalties for registration violations and vehicle size and weight violations. In each State, financial support for the police agency is determined by the legislature. Finances may come from highway-user revenues, general fund revenues, or a combination of both.

Highway-user Revenue

Twenty-one State police agencies receive their entire support from highway-user revenues. In some agencies these moneys are allocated from a readily identified source. For example, in Ohio, the Department of Highway Safety gets an appropriation from motor-fuel tax collections and operator and chauffeur license fees. In other agencies the source of funds is not known. In Arizona, the Highway Patrol receives an appropriation from the State Highway Fund, which contains revenue from various highway-user taxes. Of these 21 States, 16 have subordinate agencies. The subordinate agencies include six departments of public safety, five highway departments, three motor vehicle departments, and two other departments. Five States have independent agencies that include two State police and three highway patrols.

General fund revenue

General fund appropriations support State police agencies in 16 States. Taxes, fees, and often highway-user revenues are deposited in the State general fund. The Bureau of Public Roads considers that the general fund appropriation by the police agency was derived from highway-user revenues to the extent that the user revenues do not exceed the appropriation.

In Alabama, Georgia, Mississippi, New Mexico, Oregon, and Pennsylvania, certain highway-user revenues are deposited into the State general fund or into a special fund created within the general fund. These revenues are appropriated by the legislature for police support or are used to reimburse the general fund for moneys previously allocated for police support. The basic difference between these States and the other 10 State agencies supported by general fund appropriations is that the source of financial support for police activities is identified as highway-user revenues. Eight of the 16 State agencies in this group are departments of public safety, two are highway departments, and six are independent State police agencies.

Table 8.—Police agency receipts for traffic and related functions, 1964

[*Thousands of dollars*]

State	Imposts on highway users			Tolls	Total highway-user revenues	General fund appropri-ations	Miscel-laneous receipts	Total receipts
	Dedicated revenues		Commin-gled high-way-user revenues					
	Motor fuel taxes	Motor vehicle revenues						
Alabama						4, 868	596	5, 464
Alaska						657		657
Arizona			6, 519		6, 519			6, 519
Arkansas		1, 810			1, 810			1, 810
California		43, 211			43, 211			43, 211
Colorado			4, 505		4, 505		109	4, 614
Connecticut			4, 469		4, 469			4, 469
Delaware				153	153	1, 388		1, 541
Florida				619	619	8, 054		8, 673
Georgia						5, 943		5, 943
Hawaii[1]								
Idaho			1, 608		1, 608			1, 608
Illinois		13, 450		538	13, 988			13, 988
Indiana			5, 796	556	6, 352	553		6, 905
Iowa						3, 439		3, 439
Kansas			1, 274	303	1, 577			1, 577
Kentucky			3, 652	101	3, 753	1, 822		5, 575
Louisiana		2, 879	159	2	3, 040	4, 158		7, 198
Maine			1, 490	12	1, 502	117	32	1, 651
Maryland		8, 521		182	8, 703			8, 703
Massachusetts			5, 310	528	5, 838			5, 838
Michigan						8, 980		8, 980
Minnesota			5, 255		5, 255			5, 255
Mississippi						5, 697		5, 697
Missouri			7, 466		7, 466			7, 466
Montana		126			126	1, 655		1, 781
Nebraska						2, 041		2, 041
Nevada			1, 230		1, 230			1, 230
New Hampshire			1, 133	104	1, 237		44	1, 281
New Jersey				2, 439	2, 439	6, 188		8, 627
New Mexico						2, 711		2, 711
New York				2, 615	2, 615	17, 117		19, 732
North Carolina			6, 117		6, 117			6, 117
North Dakota[2]								
Ohio	9, 121	1, 807		1, 029	11, 957			11, 957
Oklahoma		522		262	784	3, 058		3, 842
Oregon						3, 589		3, 589
Pennsylvania				2, 117	2, 117	15, 004		17, 121
Rhode Island						788		788
South Carolina			3, 774		3, 774			3, 774
South Dakota			1, 181		1, 181			1, 181
Tennessee						4, 044	848	4, 892
Texas		3, 521	7, 404	137	11, 062			11, 062
Utah	1, 854	666			2, 520			2, 520
Vermont			713		713	319		1, 032
Virginia		7, 945		226	8, 171		188	8, 359
Washington		8, 745			8, 745		26	8, 771
West Virginia		97		171	268	1, 417		1, 685
Wisconsin		3, 712			3, 712			3, 712
Wyoming			883		883			883
TOTAL	10, 975	97, 012	69, 938	12, 094	190, 019	103, 607	1, 843	295, 469

[1] Not included in police study.

[2] 1964 expenditures were supported by general funds appropriated in 1963.

User and general revenues

The third method of financial support for police functions is the allocation of both highway-user and general fund revenues; 12 States use this method. In Montana and North Dakota, highway-user revenue is a small percentage of the total income. Part of the driver license fees in Montana is allocated to the police retirement fund; the North Dakota highway patrol receives a small part of the registration and other related fees for its safety program. The Maryland State Police, as a result of recent legislation, is limited to an annual appropriation of $8,250,000, paid from motor-vehicle fees. Expenditures in excess of this amount are paid from the general fund.

In five States, the amounts paid from highway-user revenues and general fund revenues are based on percentages established by law: Missouri, 90-percent highway funds, 10-percent general funds; Connecticut, Indiana, and Maine, 75-percent highway funds, 25-percent general funds; Vermont, 50-percent highway funds, 50-percent general funds. The primary purpose for the appropriation of both revenues is to support traffic-related functions with highway funds, and nontraffic or criminal-related functions with general funds.

In Oklahoma, the Department of Public Safety receives the first $112,500 collected on the registration of commercial vehicles and the same amount on overweight fees; in addition, 10 percent of the net receipts from driver license fees is allotted to the pension fund. Remaining support comes from the genera fund. In Kentucky, the Department of Public Safety has received the bulk of its support from highway-user revenues deposited in the road fund. As a result of the appropriation acts passed by the 1964 General Assembly, the major source of financial support for the Kentucky Department of Public Safety was shifted from the road fund to the general fund. For fiscal year 1964–65, the general fund appropriation amounted to 56 percent of the total appropriation; for fiscal year, 1965–66, the appropriation reached 88 percent. In Louisiana, support comes from several user revenue sources and an annual general fund appropriation. In West Virginia, the Motor Vehicle Inspection Division of the State Police is self-supporting; remaining State Police support comes from the general fund.

Of the States receiving allocations from both highway-user and general fund revenues, four State agencies are subordinates of the departments of public safety, five are independent State police, and three are independent highway patrols.

The growth of expenditures for traffic-related activities from 1946 to 1964 are divided into the three basic categories in table 6. These figures include size and weight enforcement and safety education expenses for other than traffic law enforcement agencies. For example, many highway departments perform the function of weighing vehicles and operating the port of entry stations; their expenditures in this area accounted for 60 percent of the total expenditures in 1964.

In the field of safety education, the increased spending is attributable to the current trend of financing student driver training programs with highway-user revenues and other State moneys. By the close of 1964, 24 State legislatures provided for the financial support for driver training programs. These programs accounted for 52 percent of the total expenditure shown in table 6 for safety education. Motor vehicle and State agencies provided 28 percent of the total safety education programs.

Receipts and disbursements

Disbursements by function and receipts by source of funds of State police agencies are shown in tables 7 and 8, respectively. Any differences between receipts and disbursements are the result of fund balances omitted from the tables. In table 8 receipts include toll revenues and minor miscellaneous income. In the summary columns net contributions to police agencies were considered to have been derived specifically from user revenues and miscellaneous funds and not from the total general fund. To compile figures for table 7, adjustments were made in the 1964 figures shown in table 6; expenditures for administrative functions performed by police agencies were added. The table of disbursements itemizes some functions performed by police agencies. However, every function may not be performed by every State police agency; some come under the jurisdiction of another agency.

A summary of police function financing shows that 94.2 percent of revenue allocated for traffic-related police functions was derived for highway-user revenues, 5.2 percent from miscellaneous receipts. Police function allocations from the highway-user revenues were $278.4 million with $3.6 million or 1.3 percent expended for nonhighway purposes. Total 1964 disbursements of highway-user revenues amounted to $6.3 billion with $625.7 million or 9.9 percent expended for nonhighway purposes.

Special Fees for Special Services

In some States fees are charged for special services provided by the police agency. Such services include motor vehicle inspection and driver license examinations. Of the 20 State police agencies that require periodic inspection, 5 reported this service as a separate expenditure. In each of the 5 States, except New York, the fee collected for issuing inspection stickers is allocated to the police agency. Driver license examinations are conducted by the police agency in 22 States. Examination costs and related administrative costs are shown in table 7 for 19 of these States; 3 States, Alaska, Pennsylvania, and West Virginia, did not identify these costs. In 6 of the 22 States, part or all of the revenues collected from operator and chauffeur licenses is allocated directly to the police agency. In one State, the amount allocated from these fees is used to support the Driver License Division of the Department of Public Safety, which conducts the driver examinations. In 15 States, revenue from these fees is deposited in the fund that supports the police agencies. The operator and chauffeur license fees also support driver training programs in 11 States, although such programs are not a primary function of the agency.

Police agencies in many States support traffic-related activities with highway-user revenues and nontraffic-related activities with general funds. Assuming that this method of financing is the most acceptable, the problem remains to determine the type of highway-user tax to be used for support of traffic-related functions. In 1964, Prof. Robert G. Hennes of the University of Washington, in *Memorandum on Patrol Financing,* reported his study of the problem in the financing of the Washington State Patrol.

Following the practice of allocating costs in proportion to the benefits received, Professor Hennes first estimated the percentage of patrol costs incurred for highway functions and second the percentage of patrol costs incurred for related functions. Using data from a previous study, he determined that 51 percent of the patrol's time was spent on traffic patrol, and that the remainder was equally divided between traffic-related functions and functions more closely related to the number of vehicles than to the distance they traveled, such as vehicle inspection, licensing, weight control, and so on. The separation of the total functions into these components provided the basis for assigning financial support. The traffic-related costs, constituting about 75 percent of the patrol's highway-related function costs, was to be collected from road-users in proportion to vehicle-miles traveled. Fees would be allocated to each vehicle class by multiplying the number registered by the miles traveled, and converting the result into the percentage of total miles traveled for each vehicle class.

Weight control expenditures would be computed from the remaining 25 percent of the patrol's highway-related function costs and then apportioned to commercial vehicles in relation to their use of the highway. Remaining expenditures would be allocated to the three classes of vehicles at a set fee. Nonhighway costs would be financed from general fund contributions, if possible. Professor Hennes believed that the heavy truck-over-10-ton, should support about three times as much of the highway patrol budget as an automobile or light truck. But this apportionment of costs would have little effect because the number of vehicles in this class is small. Hennes also suggested that police activity might be further supported by gas tax instead of the vehicle license fee, as this revenue increases with the amount of travel, as does patrol activities.

NEW PUBLICATIONS

Freeways to Urban Development

Freeways to Urban Development, A new concept for joint development, explains and illustrates in leaflet form a new Bureau of Public Roads concept for joint urban freeway and urban development. It is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, for 15 cents a copy.

The Public Roads concept is presented in response to President Johnson's call for imaginative thinking in meeting the growing demands of the Nation's cities. Joint development offers a way for city planners to surmount the limitations of money, space, and time, and provide housing, parks, and other community facilities simultaneously with the construction of freeways. The concept is based on the economics of land acquisition and use. Bureau of Public Roads studies indicate that in some urban situations the cost of acquiring whole blocks or squares of property would be comparable to or only slightly higher than the cost of acquiring freeway rights-of-way including payment for severance damages or the remainder where only part of the property is acquired. Thus, a city could acquire land on the route of a planned freeway, sell freeway space to the highway department, and still retain land for urban development at a fraction of the cost of acquiring it alone.

The space alongside, over, and under the freeway could be used to meet any of the city's redevelopment needs. Construction of the freeway could be coordinated with other development so that new replacement housing and buildings would be available for those displaced as construction progressed; those displaced could be given priority for space in the new facilities.

As a massive nationwide freeway construction program is now underway, there is an immediate opportunity for joint urban development. The Bureau of Public Roads is ready to work with the Nation's cities, through State highway departments and other Federal agencies to achieve maximum use of the scarce urban land that is needed for highway transportation.

U.S. GOVERNMENT PRINTING OFFICE: 1966

PUBLICATIONS
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Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:
  Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.
  Supplementary Report, House Document No. 124 (1965). $1.00.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42 (1965). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61, $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

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Calibrating and Testing a Gravity Model for Any Size Urban Area (1965). $1.00.

Capacity Charts for the Hydraulic Design of Highway Culverts (Hydraulic Engineering Circular, No. 10) (1965). 65 cents.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Design of Roadside Drainage Channels (1965). 40 cents.

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Freeways to Urban Development. A new concept for joint development (1966). 15 cents.

Highway Bond Financing . . . An Analysis, 1950–62. 35 cents.

Highway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Research and Development Studies, Using Federal Aid Research and Planning Funds (1964). $1.00.

PUBLICATIONS—Continued

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Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

Traffic Assignment and Distribution for Small Urban Areas (1965). $1.00.

Traffic Assignment Manual (1964). $1.50.

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VOL. 34, NO. 6

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Public Roads

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West Potomac Park in Lincoln Memorial area, Washington, D.C., and interchange for Arlington Memorial Bridge.

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ERRATA

In the article *State Highway Patrol Functions and Financing*, in the December 1966 issue of Public Roads, *A Journal of Highway Research*, volume 34, No. 5, on page 111, column 1, the following statement was inadvertently included. "The increased mileage of divided highways is followed by increased traffic accidents and fatalities each year." The following statement is more accurate. "Traffic accidents continue to increase despite the increase in mileage of divided highways. Accident rates on divided highways are much lower than those on undivided highways."

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BUREAU OF PUBLIC ROADS
REX M WHITTON, Administrator

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 16, 1966.

Contents of this publication may be reprinted. Mention of source is requested.

Gap Utilization, A Warrant for Traffic Signal Control

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

**Reported by [1] RICHARD D. DESROSIERS,
Highway Research Engineer,
Traffic Systems Division**

Results of the research reported in this article show that a warrant for traffic signal control is feasible based on the relation of the gap availability in the traffic on major streets to the lag or gap acceptance characteristics of the driver on minor streets. Properly operated traffic signals for allocating time and space to the different movements at intersections and other street and highway locations are undoubtedly necessary, but traffic signals are not the answer to all traffic problems at intersections. The volume of traffic in itself does not determine the need for signals; distribution and utilization of available gaps in traffic flow on main streets by vehicles or pedestrians on minor streets should be of critical consideration. The methodology developed in the research reported in this article can be applied to any unsignalized intersection. To use the methodology accurately, however, it is necessary that the traffic engineer obtain data on the variations in the distribution of available gaps and characteristics of gap acceptance at the individual intersections studied.

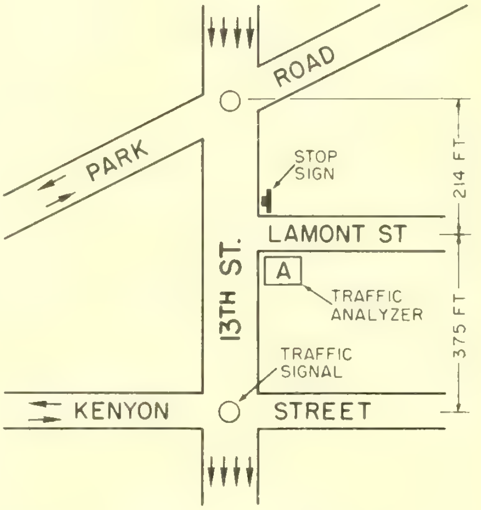

Figure 1.—Test site.

Introduction

A HIGHWAY traffic signal is a means of allocating time and space to the different movements necessary at intersections and other street and highway locations. Properly located and operated, traffic signals can serve several purposes:[2]

- They provide for orderly movement of traffic. Where proper physical layouts and control measures are used they can increase the traffic-handling capacity of the intersection.
- They reduce the frequency of certain types of accidents.
- Under conditions of favorable spacing, they can be coordinated to provide for continuous or nearly continuous movement of traffic at a definite speed along a given route.
- They can be used to interrupt heavy traffic at intervals to permit other traffic, pedestrian or vehicular, to cross.
- They represent a considerable economy, as compared to manual control, at intersections where the need for some definite means of assigning right-of-way first to one movement and then to another is indicated by the volumes of vehicular and pedestrian traffic or by the occurrence of accidents.

The necessity for and the usefulness of traffic signals is unquestionable. However, many laymen and some engineers believe that traffic signals are the answer to all traffic problems at intersections. This belief has caused public and private pressures on the practicing traffic engineer or agency with the responsibility for the traffic engineering function.

The existing warrants for traffic signal control are presented in the manual on uniform traffic control devices, page 185. The manual also includes as desirable information the number and distribution of gaps in vehicular traffic on the main street when traffic from the minor street finds it possible to use the intersection safely. This point is very important and should receive additional study. It is reasonable to assume that no traffic signal should be installed if all traffic on the minor street can be accommodated without conflict by existing gaps in the traffic on the major street. This is the basis for the major premise for the study reported here; namely, that the volume of traffic in itself may not be critical but the distribution and utilization of available gaps on the main street by vehicles from the minor street may be.

The purpose of the research was to determine whether a warrant for traffic signal control could be established by using the relation between the availability of gaps in the traffic stream on the major street and the lag and or gap acceptance by the driver on the minor street. Definition of the following terms is necessary for an understanding of the procedures and results of the research reported in this article.

Gap.—A gap is the elapsed time between arrival of successive vehicles on the main street at a specified reference point in the intersection area.

Lag. A lag is that part of a current gap remaining when a vehicle on the minor street arrives; in other words, the elapsed time at an intersection between arrival at the intersection of a vehicle on the minor street and arrival of the next vehicle on the main street.

Gap or lag acceptance.—A gap or lag is either accepted or not accepted by the driver of the vehicle on the minor street. A lag is accepted if the vehicle on the minor street crosses the intersection or enters the main street before arrival of the next vehicle on the main street. A gap is accepted if the vehicle on the minor street crosses or enters between two vehicles on the main street.

Gap utilization.—Gap utilization refers to the probability that the length of an available gap in the traffic on the main street is the same as the gap length acceptable to the driver of the vehicle on the minor street. That is, the gap is both available and acceptable, and the result is a gap in the traffic on the main street being used by a vehicle on the minor street.

To accomplish the objective of the study, information had to be obtained on gap distribution in a traffic stream and the characteristics of the driving population as to gap acceptance and to relate this information by use of acceptable statistical probability techniques. The general procedure used is described in the following paragraphs. Gap distribution (availability) information was obtained by collection of field data on 13th Street NW., Washington, D.C. Driver characteristics on gap acceptance were obtained from a survey of the existing literature.

The probability of the availability or the acceptance of a gap of any given size was determined by using the probability distributions developed for gap availability and acceptance. The probability of a gap of a given length being *both* available and accepted was computed by taking the product of the two individual probabilities. By summing the product probabilities for all turning movements and considering all gap lengths, the percentage of total gaps on the main street that would be expected to be utilized by vehicles from the minor street in 1 hour were established. This percentage was converted into an expected number of gaps utilized, which is the same as the number of vehicles that can be accommodated on the minor street when the intersection has no traffic signal control.

---

[1] Past Presidents Award Paper presented at the 36th annual meeting of the Institute of Traffic Engineers, Cincinnati, Ohio, Oct. 11, 1966.

[2] *Manual on Uniform Traffic Control Devices for Streets and Highways*, by the National Joint Committee on Uniform Traffic Control Devices; published by U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., 1961, p. 155.

Conclusions

The results of the research reported here show that a warrant for traffic signal control is feasible based on data on the availability of gaps in the traffic stream on the major street and the gap acceptance characteristics of the drivers on the minor street. The methodology developed in the study reported here can be applied to any intersection that does not have a traffic signal.

Gap availability and acceptance by the driver vary in relation to geometrics, sight obstructions, type of entering maneuver, traffic volume speed of the vehicles on the main street, and sequence of gap formation during periods of heavy traffic. The extent of importance of this variation on the resultant number of gaps available or accepted is unknown. Therefore, the methodology requires that the user obtain data on the distribution of available gaps and the drivers characteristics on gap acceptance at each intersection studied. Future research should include an examination of the extent and importance of all parameters on the resultant answers; that is, the number of gaps utilized. Relations between the variables could be presented in a graphic form similar to the one used in the *Highway Capacity Manual, 1965*, Highway Research Board Special Report 87. Presentation in this manner would permit the practicing traffic engineer to enter inventory data in the manual and thus establish the expected number of gaps utilized. Such simplicity of application would encourage more use of vehicular gap characteristics when traffic signal requirements are being considered. Hopefully, the installation of unneeded traffic signals would thereby be reduced.

Test Site and Equipment

Thirteenth Street, an arterial street in Washington, D.C., with a progressively timed signal system was used as the study site, which is shown in figure 1, to obtain the gap availability information required. This street is a 4-lane arterial on which traffic is 1-way southbound from 7 to 9:30 a.m.; 1-way northbound from 4 to 6:30 p.m. and 2-way for the remainder of the day. Data used in the study were collected only during the morning rush period when all traffic was southbound on each of the 4 lanes. Traffic during the study period had essentially the same composition at all times data were collected: passenger cars, 97 percent; buses, 1 percent; and trucks, 2 percent. The traffic signals were coordinated for a 27 m.p.h. speed of progression. An 80-second cycle and a through band for traffic of approximately 55 seconds were in effect.

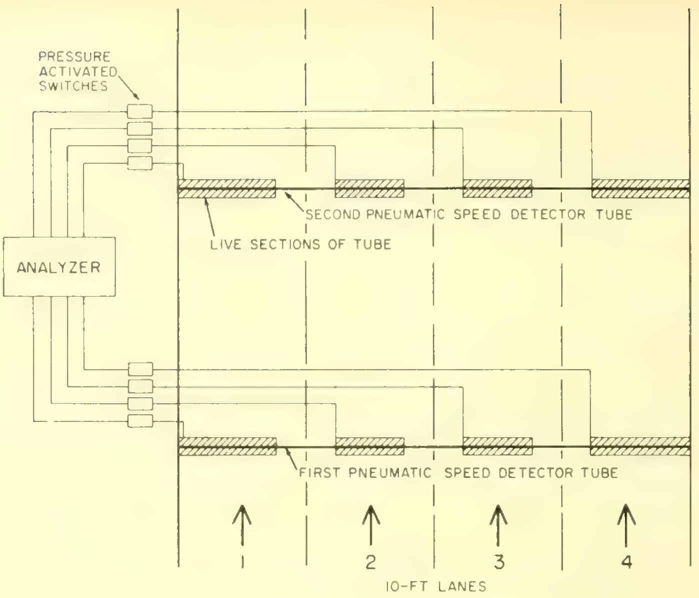

Figure 2.—Layout of speed detection equipment.

Speed, volume, and headway information were obtained with the Bureau of Public Roads traffic analyzer. The traffic analyzer is a mobile unit containing an assembly of equipment that provides traffic information by an automatic digital recording of speed, time headway, volume, and lateral placement data at a single point on a street or highway. All capabilities of the analyzer, except for recording lateral placement, were used. The equipment consists essentially of four adding machine recorders that have specially designed solenoids mounted over the keys, a digital timer to register the time of day for each recording, and a speed timing device. Speed is detected by use of two pneumatic pressure tubes that form a speed trap. The sequence of operation is initiated when a vehicle actuates the first speed detector tube. This electrical contact starts the speed timer and activates the placement detector for that location. When the front wheels of the vehicle pass over the second tube of the speed detector, the reading in units of one-hundredths of a second is recorded, the reading of the timer in ten-thousandths of an hour is transferred to the recorder, and a control signal activates the adding machine, which prints these data.

A typical layout of the speed detectors is shown in figure 2. Two pneumatic tubes were spaced 15 feet apart. Speed by lane was obtained by alternating live and dead areas—areas of detection and nondetection, respectively—along the pneumatic tubes. A dead area of 5 feet, 2½ feet on either side of the lane line, was provided between adjacent lanes. The live area was extensive enough for positive detection of all vehicles in the lane, but small enough to eliminate false actuations by vehicles traveling in adjacent lanes. Detection was obtained and signals were transmitted to the analyzer by pressure actuated switches attached to each live area.

A section of a typical 14-column printing tape output is shown in figure 3. Time of day in ten-thousandths of an hour was recorded in columns 1 through 4. Columns 5 through 10, in which lateral placement data are recorded, were not used in the study reported here. The time in one-hundredths of a second required for the vehicle to traverse the trap distance was recorded in columns 11 and 12. Column 13 was used to show whether the vehicle originated above Monroe Street, off Monroe Street, off Park Road, or off Lamont Street. The last column was used to

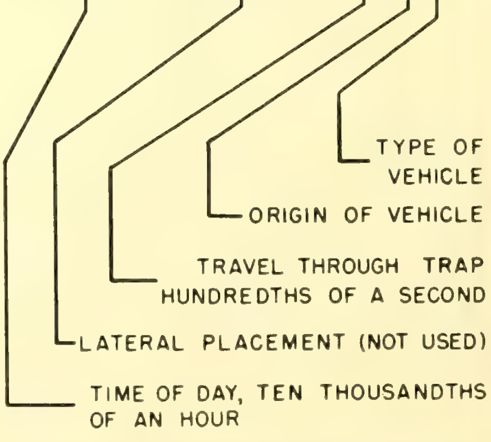

Figure 3.—Typical printed output tape.

code vehicle type. Coding of vehicle origin and type was performed manually; an operator was stationed at each recorder to observe and classify each vehicle and to depress the key for the appropriate code light. An explanation of the coding in columns 13 and 14 is given in table 1.

Analysis

The analysis of distribution of gaps in a traffic stream cannot be separated from volume and speed characteristics because distance between vehicles decreases with an increase in volume and increases with an increase in speed. This interrelation precludes the analysis of any of the three items without discussing the characteristics of the remaining two as a base or background condition.

Traffic volume

Traffic volume during the study period is reported in figure 4. Volume is shown for each 12-minute period from 7:18 to 9:30 a.m. The peak 12-minute volume of 710 vehicles occurred during the period from 8:06 to 8:18 a.m. The traffic volume by lane is reported in figure 5. Each of the center lanes carried considerably more vehicles, in all time periods, than the two-curb lanes. Traffic in the right-curb lane was especially low for all time periods studied. Even in the peak hour this lane carried only about one-half to two-thirds the traffic volume carried by the center lanes.

Table 1.—Traffic analyzer coding

Code:	COLUMN 13
0	Vehicle on 13th St. north of Park Rd.
1	First vehicle clearing green period on 13th St.
2	Vehicle entering 13th St. from Lamont St.
3	Vehicle entering from Park Rd., eastbound.
4	Vehicle entering from Park Rd., westbound (left turn).
9	First vehicle clearing green period on Park Rd.

Code:	COLUMN 14
0	Passenger car.
1	Pickup or panel truck, single tires on rear.
2	Single-unit truck, dual tires on rear.
3	Single-unit truck, tandem axle, dual tires on rear.
4	3-axle truck tractor semitrailer, dual tires on rear.
5	4- or 5-axle truck tractor semitrailer, dual tires on rear.
7	Towing vehicle or passenger car with trailer.
8	Motorcycle or scooter.
9	Bus (commercial).

Table 2.—Speed data by lane

Lane	Speed		
	Mean	Standard deviation	85th percentile
	M.p.h.	*M.p.h.*	M.*p.h.*
1	23.0	4.74	27.5
2	24.6	5.25	29.0
3	24.0	5.01	28.2
4	24.5	5.67	29.3

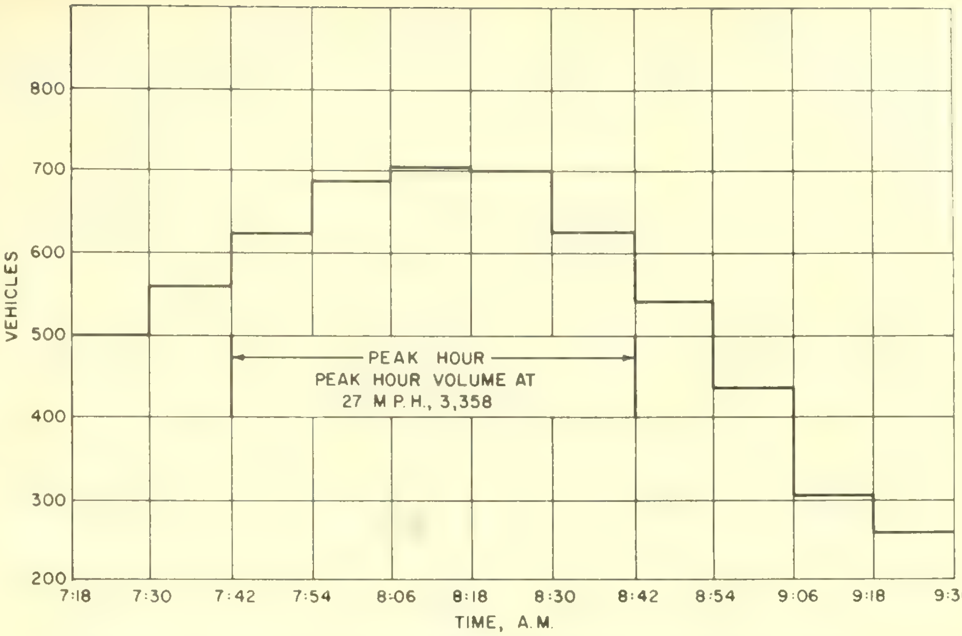

Figure 4.—Average number of vehicles by 12-minute periods, all 4 lanes.

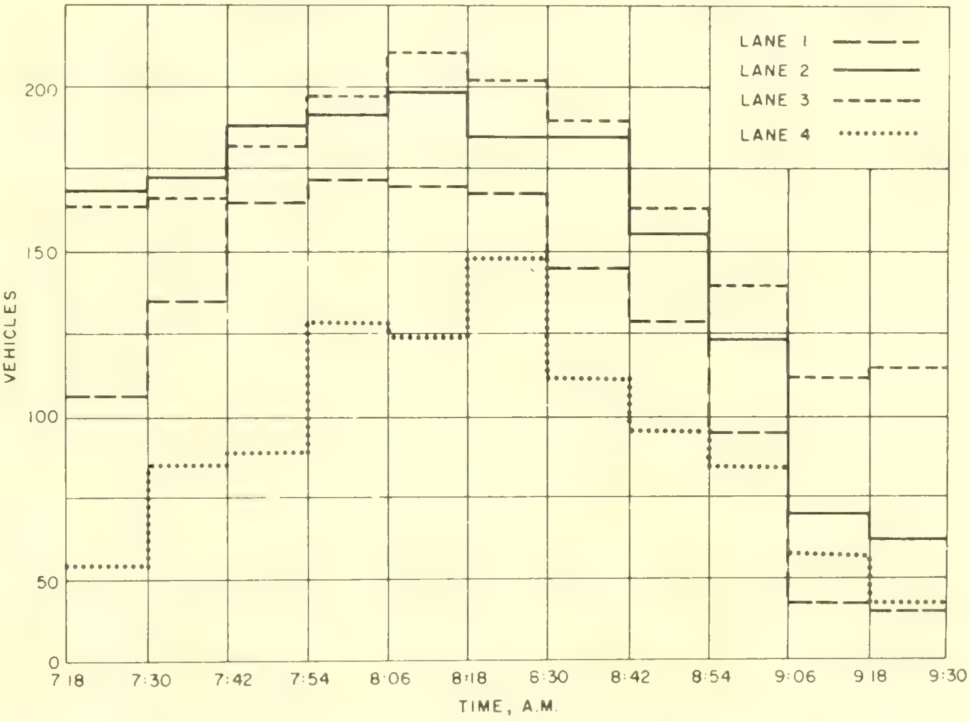

Figure 5.—Average number of vehicles per lane by 12-minute periods, southbound.

Vehicle Speed

The distribution of spot speeds is shown in figure 6. The speeds of all vehicles during the period from 7:30 to 9:30 a.m. are included. The mean speed was 23.9 m.p.h. and the standard deviation was 5 m.p.h.; at this mean speed and standard deviation, 67 percent of all vehicles can be expected to travel at speeds from 18.9 to 29.9 m.p.h. and 95 percent can be expected to travel at speeds from 13.9 to 33.9 m.p.h. The 85th percentile speed, a speed often used by traffic engineers to establish speed limits, was 28.4 m.p.h. Speed characteristics by lane are shown in table 2. The speed in lane 1—the far left lane when travel is southbound—was significantly slower (5-percent level) than in the other 3 lanes, where speeds were nearly identical. The differences in mean speed, 85th percentile speed, and standard deviation for the different lanes were small.

Gap Availability

The distribution of all gaps observed during the study period is presented in figure 7. The majority of the gaps (83.5 percent) was in the range of 0.5 to 1.5 seconds, 6.5 percent of the gaps were in the range of 1.5 to 2.5 seconds, and headway for only 10 percent of the gaps was more than 2.5 seconds. From these figures, it can be concluded, and rightfully so, that few acceptable gaps would

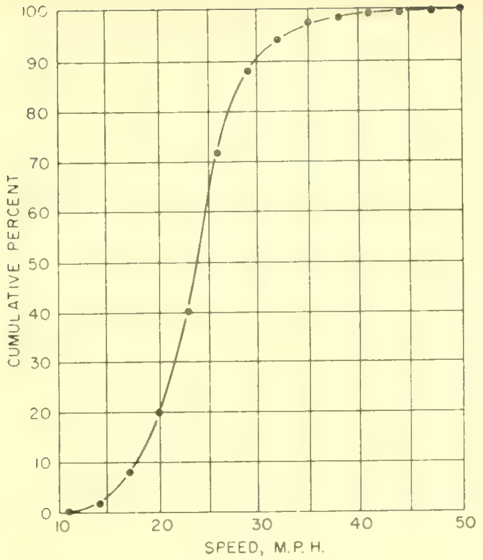

Figure 6.—Cumulative distribution of spot speeds on 13th Street, midway between Park Road and Kenyon Street.

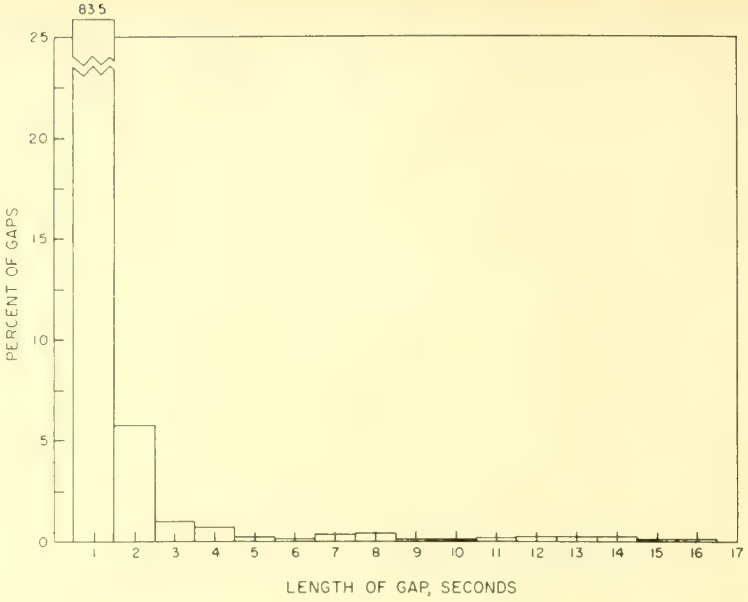

Figure 7.—Percentage of available gaps as a function of gap size.

be available to drivers of vehicles on the minor street trying to enter the traffic stream on the main street. If the distribution of gaps is considered at different vehicle flow rates, however, a somewhat different opinion may be formed. The cumulative gap distribution for five flow rates is shown in figure 8. The shape of the curves and the increased steepness as flow rate increased was not unexpected. The magnitude of these differences, however, is worthy of note. At a flow rate of 1,000 to 1,050 vehicles per hour per lane, 84 percent of the gaps were 0.5 second or less in length, 90 percent 1.0 second or less, and 96 percent 2 seconds or less. For a flow rate of 300 to 350 vehicles per hour per lane, distribution of gap length would be 40 percent, 6 percent and 80 percent, respectively.

The platooning effect of vehicles has often been mentioned in the literature but few data have been presented to determine the magnitude of this effect. A comparison of the gap distribution for vehicles within and between platoons is presented in figure 9 for five flow rates. For purposes of the study described here, vehicles between platoons are defined as those vehicles that entered the main street while traffic on the main street was stopped at a traffic signal (minor movement phase) and vehicles that entered the main street from uncontrolled minor streets between groups of vehicles. For the vehicles within platoons, a curve was drawn for each of the five flow rates. Because of the overlap in the data points for certain flow rates, only two curves are shown for the vehicles between platoons. One curve represents flow rates for 300 to 500 vehicles per hour per lane and the other represents flow rates for 650 to 1, 050 vehicles per hour per lane.

The separation between gap distributions for vehicles with and between platoons is a measure of platooning effect. Even at the largest rate of flow, 45 percent of the gaps between platoons would exceed 2 seconds. Thus, the presence of a traffic signal would cause two distinctly different gap distributions to occur at a downstream uncontrolled intersection. The within platoon distribution would have a large concentration of very short gaps, and the between platoon distribution would have long gaps. Platooning provides more gaps acceptable to drivers from the minor street; the absence of platooning provides fewer acceptable gaps because vehicle gaps are more uniformly distributed. The development of a gap utilization warrant for traffic signal control, which is discussed subsequently, was based on knowledge of the effect of platooning. However, note that the distributions referred to will vary in relation to cycle length, split, distance to the nearest upstream traffic signal, and possibly geometric conditions; but, the basic relation should remain unchanged.

Gap Acceptance

Data on gap acceptance were not collected in the research because satisfactory information could be obtained from a composite of the findings reported in the available literature. The results of five of the more significant research projects are shown in figure 10. The differences between the reported results are not fully understood, but some evidence exists that gap and/or lag acceptance distributions would vary with geometrics, sight obstructions, type of entering maneuver, volume, speed of the vehicles on the main street, and sequence of gap formation during periods of heavy traffic. The researchers on the five projects studied intersections that differed substantially in these characteristics. Despite these differences, the results are reasonably consistent.

A composite of the results of these five projects is shown in figure 11. This composite is essentially an approximate average of the results reported for the five projects; however, no gap of less than 2 seconds long was considered acceptable. Raff reported acceptance of gaps less than 2 seconds but did not report conditions under which these acceptances occurred. This composite curve of gap acceptance is used in conjunction with the gap availability distribution curve as an example of the application of the methodology developed in the research for this report.

Traffic Signal Warrant Methodology

As with conventional traffic signal warrants, each approach on the minor street must be analyzed separately to determine which one is critical. Gap utilization logic was developed for both approaches on the minor street for the intersection shown in figure 12. The intersection is the crossing of a 4-lane, 1-way urban arterial and a 2-lane, 2-way street. The equations developed subsequently are restricted to this geometric configuration but the general methodology could be used to develop equations for any other geometric conditions.

Gap utilization on east approach

For gap utilization by vehicles on the east approach three probabilities were considered: (1) The probability of a gap being utilized by a vehicle crossing the main street, (2) the probability of a gap being utilized by a vehicle turning left, and (3) the probability of a vehicle crossing from the west approach interfering with a vehicle turning left from the east approach. The latter must be subtracted from the sum of the other two to obtain the net percentage of gaps utilized. This can be represented algebraically as:

$$P(U_t)=P(E_T)+P(E_L)-P(W_I) \qquad (1)$$

Where,

$P(U_t)$ = The probability that a gap of size t seconds will be utilized.

$P(E_T)$ = The probability that a gap of size t will be utilized by a vehicle crossing the main street.

$P(E_L)$ = The probability that a gap of size t will be utilized by a vehicle turning left.

$P(W_I)$ = The probability that a vehicle crossing from the west approach will interfere with a vehicle turning left.

The probability of gap utilization by the crossing vehicle can be determined by the equation:

$$P(E_T) = T_E[P(x)][P(y/x)] \qquad (2)$$

Where,

T_E = The percentage of the vehicles from the east approach that are through vehicles, expressed as a decimal.

$P(x)$ = The probability of a gap of size t being available in the traffic stream on the main street.

$P(y/x)$ = The probability of a gap of size t being accepted by the driver of a vehicle on the minor street.

Assuming that turning maneuvers can be performed in 2 lanes and some number of such maneuvers will be executed if gaps are available in the 2 lanes nearest the turning vehicles even if main street vehicles are present in the 2 farthest lanes, the probability of a gap being utilized by a vehicle turning left can be determined from the equation:

$$P(L_T) = L_E[P(V)][P(y/v)] \qquad (3)$$

Where,

L_E = The percentage of minor street traffic turning left, expressed as a decimal.

$P(V)$ = The probability of a gap of size t being available in lanes 1 and 2 combined.

$P(y/v)$ = The probability of a gap of size t in lanes 1 and 2 being accepted by a vehicle from the minor street that is turning left.

The probability that a crossing vehicle from the west approach will interfere with a vehicle turning left can be obtained from the equation:

$$P(W_I) = (T_W[P(x)][P(y/x)])(L_E[P(v)][P(y/v)]) \qquad (4)$$

Where,

T_W = The percentage of through vehicles from the west approach of the minor street, expressed as a decimal.

Substitutions in equation (1) yield the percentage of gaps of size t that would be utilized:

$$P(U_t) = T_E[P(x)][P(y/x)] + L_E[P(v)][P(y/v)] - (T_W[P(x)][P(y/x)])(L_E[P(v)][P(y/v)]) \qquad (5)$$

Gap utilization on west approach

The gap utilization by vehicles from the west approach also involves three probabilities: (1) The probability of a gap being utilized by a vehicle crossing the main street, (2) the probability of a gap being utilized by a vehicle turning right, and (3) the probability of a

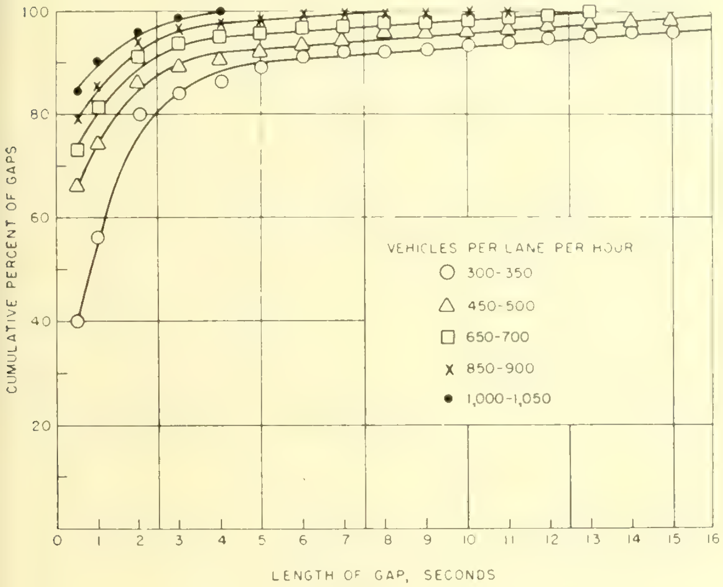

Figure 8.—Cumulative percentage of gaps for within and between platoons as a function of flow rate, all 4 lanes.

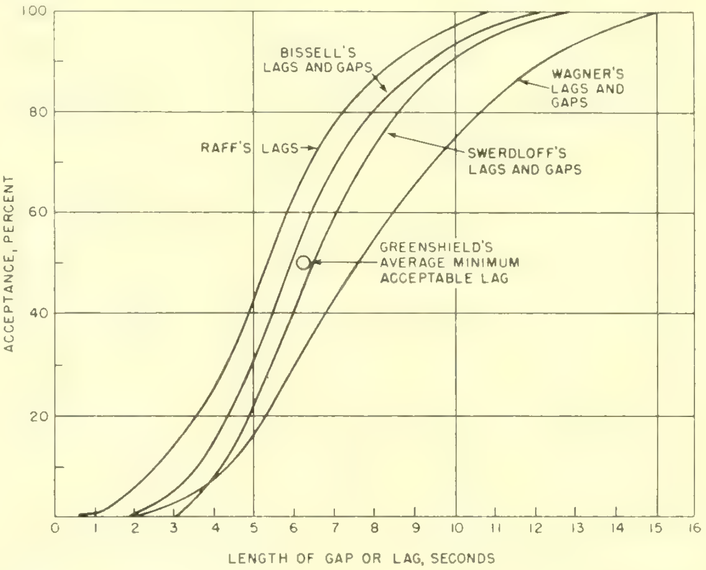

Figure 10.—Gap acceptance distributions obtained from the literature.

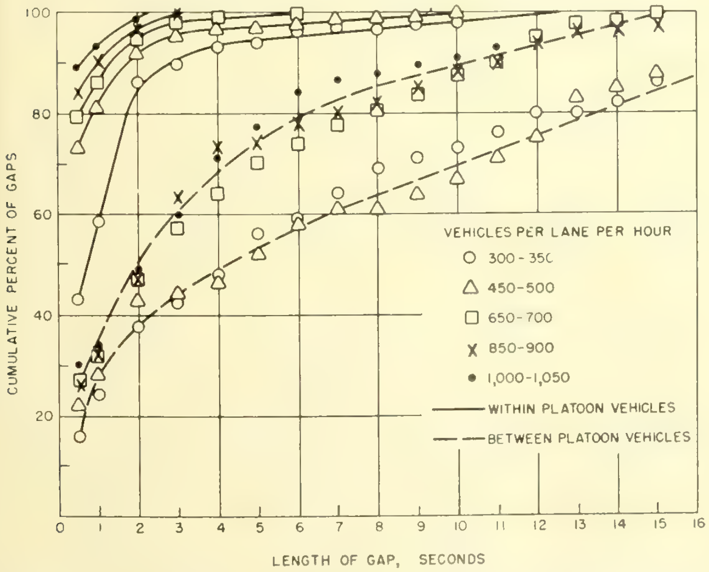

Figure 9.—Cumulative distribution of gaps for within and between platoons as a function of flow rate, all 4 lanes.

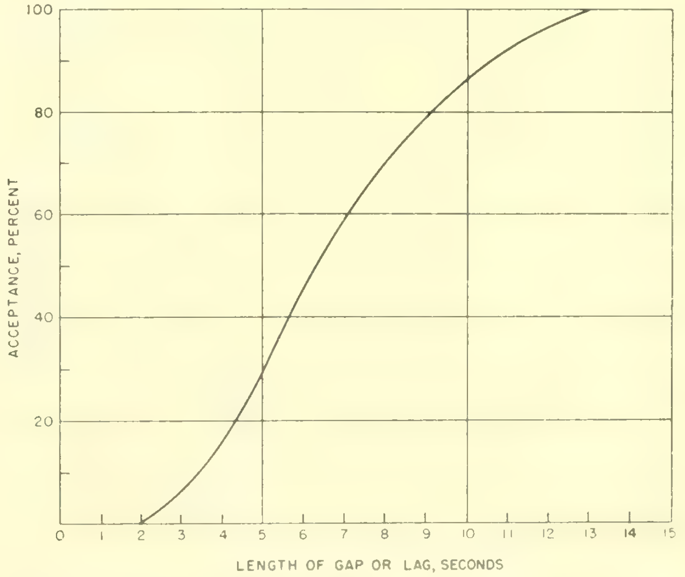

Figure 11.—Gap acceptance distribution used for the study reported in this article.

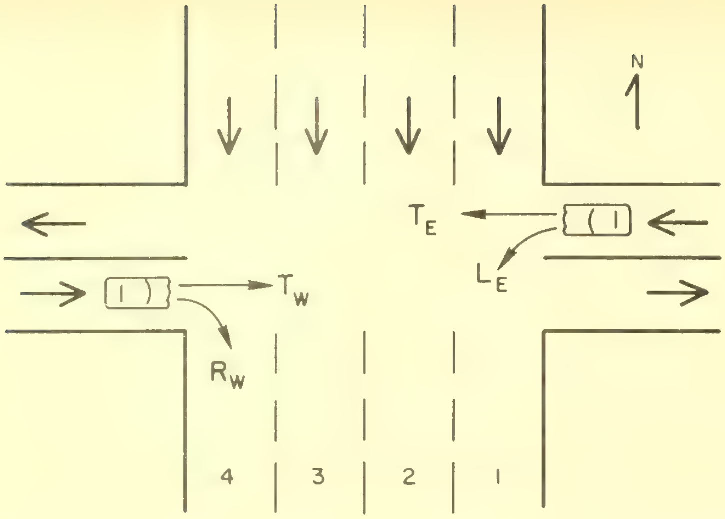

Figure 12.—Intersection schematic for sample problem calculations.

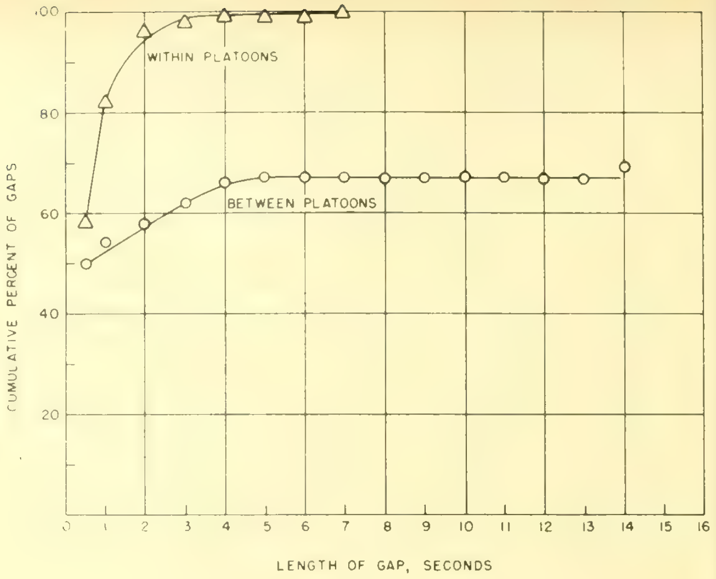

Figure 13.—Cumulative distribution of gaps for within and between platoons as a function of flow rate, lanes 1 and 2.

vehicle turning left from the east approach interfering with a vehicle crossing from the west approach. By using the same techniques as for the east approach the resultant equation for the percentage of gap utilized by vehicles from the west approach is:

$$P(U)_W = T_W[P(x)][P(y/x)] + R_W[P(W)][P(y/w)] - (T_W[P(x)][P(y/x)])(L_E[P(v)][P(y/v)]) \quad (6)$$

Where,

R_W = The percent of traffic that is turning right from the minor street expressed as a decimal.

$P(W)$ = The probability of a gap of size t being available in lanes 3 and 4 combined.

$P(y/w)$ = The probability of a gap of size t in lanes 3 and 4 being accepted by a vehicle that is turning right from the minor street.

The equations developed for the east and west approaches yield the probable percentage of gaps of size t that would be utilized. By summing all values of t, the probable percentage of all gaps utilized can be obtained. This can be converted to an expected number of gaps by taking the product of the probable percentage utilized and the total number of gaps available in the main stream. The latter is an estimate of the number of vehicles that could be accommodated on the side street when the intersection has no traffic signal control.

Sample Calculation

A clearer understanding of the application of the technique may be obtained from a sample calculation. The objective of the calculation is to determine the number of vehicles that can be accommodated in 1 hour on the east approach of the intersection shown in figure 12 when no traffic signal control is present; that is, to determine the number of gaps on the main street that would be expected to be utilized by vehicles on the east approach. Calculations are based on the assumptions that 10 percent of the vehicles on each approach will turn and that traffic volume on the main street is 1,025 vehicles per lane per hour. In practice the information in the two assumptions can be obtained by field study or possibly from existing data on file in the public agency having responsibility for the traffic function. The equation for gap utilization by vehicles on the east approach is:

$$P(U) = T_E[P(x)][P(y/x)] + L_E[P(v)][P(y/v)] - (T_W[P(x)][P(y/x)])L_E[P(v)][P(y/v)]) \quad (7)$$

For ease of application, format of table 3 has been established for calculation of the sum of the product terms $[P(x)][P(y/x)]$ and $[P(v)][P(y/v)]$. The sums of the product terms have been calculated separately for the gaps within and between platoons. This permits conversion from probability of utilization to expected number of gaps utilized by vehicles from the minor street, which will become clearer as the solving of the sample problem is continued. The detailed calculations of these terms are included in a subsequent paragraph. The probability of utilization of gaps within platoons is determined by:

$$P(U)_W = T_E[P(x)_W][P(y/x)] + L_E[P(v)_W][P(y/v)] - (T_W[P(x)_W][P(y/x)](L_E[P(v)_W][P(y/v)]) \quad (8)$$

solved as:

$$P(U)_W = 0.9(0.002) + 0.1(0.019) - (0.9)(0.1)(0.002)(0.019)$$
$$= 0.0018 + 0.0019 - (0)$$
$$= 0.0037\text{, rounded to } 0.004.$$

The probability of utilization of gaps between platoons is obtained from the same equation (8) except that $P(X)_B$ and $P(V)_B$ are substituted for $P(X)_W$ and $P(V)_W$, respectively, so that

Table 3.—Product terms for use in sample problem

Gap length t	Available gaps		Acceptance of a gap of size t $P(y/x)$	[1] $[P(x)_W]$ $[P(y/x)]$	[2] $[P(x)_B]$ $[P(y/x)]$	Available gaps		Acceptance of gap of size t $P(y/v)$	[3] $[P(v)_W]$ $[P(y/v)]$	[4] $[P(v)_B]$ $[P(y/v)]$
	Within $P(x)_W$	Between $P(x)_B$				Within $P(v)_W$	Between $P(X)_B$			
Seconds										
0.0-0.9	0.58	0.50	0	0	0	0.90	0.30	0	0	0
1.0-1.9	0.24	0.04	0	0	0	0.06	0.14	0	0	0
2.0-2.9	0.14	0.04	0.05	0.007	0.002	0.04	0.12	0.05	0.002	0.006
3.0-3.9	0.02	0.04	0.13	0.003	0.005		0.10	0.13		0.013
4.0-4.9	0.01	0.04	0.23	0.002	0.007		0.10	0.23		0.023
5.0-5.9	0	0.01	0.37	0	0.004		0.06	0.37		0.022
6.0-6.9	0	0	0.52	0	0		0.05	0.52		0.026
7.0-7.9	0.01	0	0.65	0.007	0		0.01	0.65		0.007
8.0-8.9		0	0.75	0	0		0.00	0.75		0
9.0-9.9		0	0.83		0		0.02	0.83		0.017
10.0-10.9		0	0.91	----	0		0.02	0.91		0.018
11.0-11.9		0	0.97		0		0.02	0.97		0.019
12.0-12.9		0	1.00		0		0.03	1.00		0.010
13.0-13.9		0	1.00		0.010		0.01	1.00		0.010
14.0-14.9		0.02	1.00		0.020		0.01	1.00		0.010
15.0 and longer		0.30	1.00		0.300		0.01	1.00		0.010

[1] Total is 0.019. [2] Total is 0.348. [3] Total is 0.002. [4] Total is 0.191.

$$
\begin{aligned}
P(U)_B = 0.9(0.191)+0.1(0.348) \\
-(0.9)(0.1)(0.191)(0.348) \\
=0.1719+0.0348-0.0126 \\
=0.2141\text{, rounded to } 0.214
\end{aligned}
$$

The probability of utilization of gaps within and between platoons determined, the probable number of gaps utilized by vehicles on the minor street can be obtained from the product of the appropriate probability and volume. For example, the probable number of gaps within platoons that would be utilized is the product of the within platoon probability of utilization and the within platoon volume. Similarly, the probable number of gaps between platoons that would be utilized is the product of the between platoon probability of utilization and the platoon volume. The total number of gaps on the main street that would be expected to be utilized by vehicles on the east approach is obtained by summing the corresponding within and between platoon gaps. The probable number of gaps within platoons utilized by vehicles on the minor street is:

$$
\begin{aligned}
N_W = P(U)_W(V_W) \\
=(0.004)(3{,}484)=14 \qquad (9)
\end{aligned}
$$

The probable number of gaps between platoons utilized by vehicles on the minor street is:

$$
\begin{aligned}
N_B = P(U)_B(V_B) \\
=(0.214)(615)=132 \qquad (10)
\end{aligned}
$$

Then, the total number of gaps expected to be utilized by vehicles on the east approach would be:

$$
\begin{aligned}
N = N_W + N_B \\
=14+132=146 \qquad (11)
\end{aligned}
$$

Thus 146 is analogous to the number of vehicles that can be accommodated in 1 hour on the subject approach when the intersection has no traffic signal control. This does not mean that 146 vehicles from the minor street could be accommodated at *any* intersection where traffic volume is 1,025 vehicles per hour per lane (4 lanes). The number accommodated at *this* intersection is large because of the platooning effect caused by an upstream traffic signal. In the development of this analysis, it has been assumed that (1) vehicles are continuously present on the minor street and (2) no allowance has been made for multiple acceptances of a gap. The former is the same concept employed in determining the capacity of an intersection. The results of the latter may be an actual accommodation of a slightly larger number of vehicles on the minor street than would be expected from the methodology developed in the research discussed here.

Determination of Sum of Product Terms for Sample Problem

Table 2 has been structured to simplify the calculations for the sample problem. A listing of the terms that appear in the table and their sources are: $P(x)_W$, figure 9; $P(x)_B$, figure 9; $P(y/x)$, figure 11; $P(v)_W$, figure 13; $P(v)_B$, figure 13; and $P(y/v)$, figure 11.

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NEW PUBLICATIONS

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds

The 1966 issue of *Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds* is now for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, for $1. The studies listed are those approved in the Office of Research and Development, Bureau of Public Roads, for fiscal year 1967 and calendar year 1966 as of July 1, 1966.

The information has been grouped by 8 major technical goals of the National Program and 27 projects, that contribute data toward solutions of the major goal. The eight major technical goals are: Definition of Underlying Requirements for Highway Transport; Analytic Definition of Complex Traffic Movements; Analysis of Essential Components of Highway Transport; Development of Methods for Reliable Forecasting of Demand for Highway Transport; Development of Methods for Increased Capacity, Control, and Safety in Traffic Movement; Development of Techniques for More Precise Structural Design and Incorporation of New Materials and Structural Concepts; Development and Application of New Technology to Location, Design, Construction, and Maintenance Processes; and staff, administrative contract, or HPR projects of major local, regional or national importance. Data are also presented on the objective of each study, the conducting agency, and the funding for each study.

This publication also contains information on active projects in the American Association of State Highway Officials National Cooperative Highway Research Program (NCHRP) These projects are financed from a pool of funds consisting of 5 percent of the 1½-percent Federal-aid funds of the participating State highway departments, sponsored by AASHO and administered by the Highway Research Board of the National Academy of Sciences.

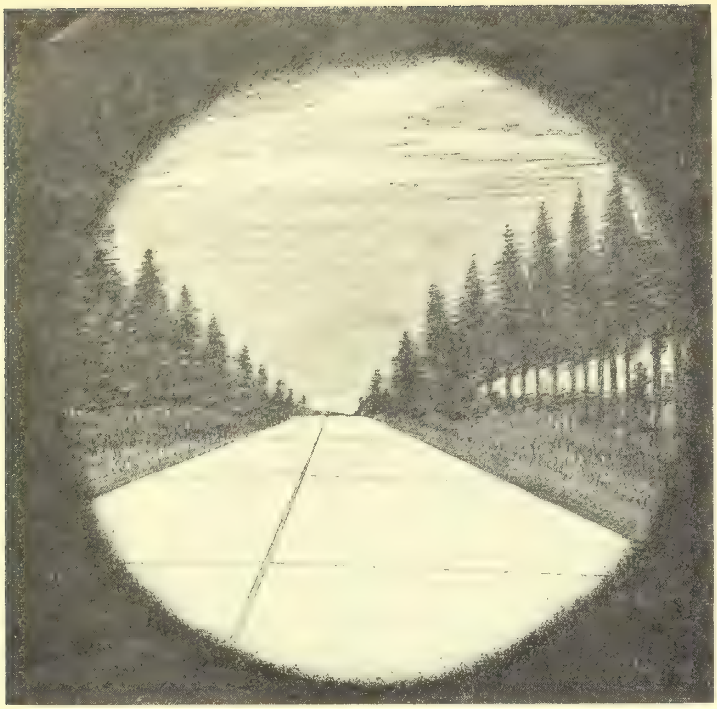

Figure 1.—Foveal view of visual field.

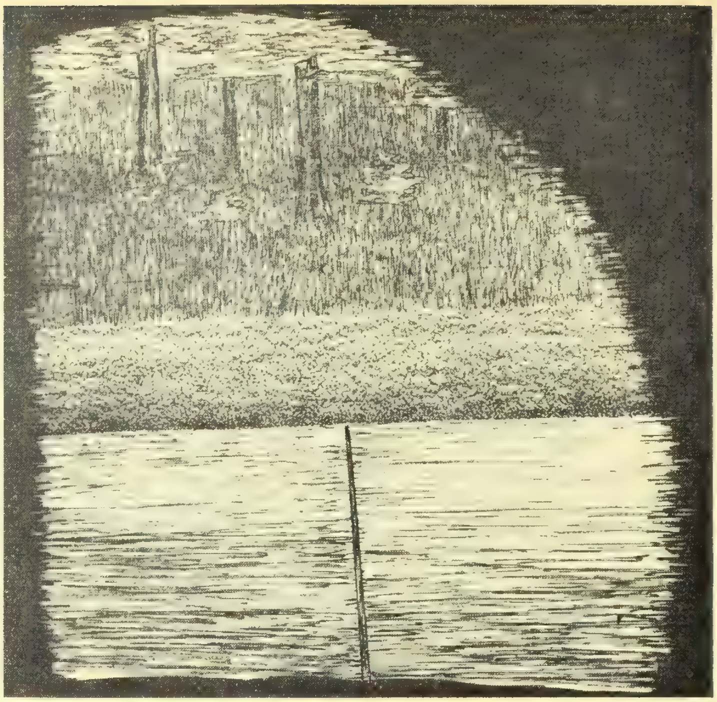

Figure 2.—Peripheral view of visual field.

Vehicle Speed Estimation from Visual Stimuli

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

**Reported by SANTO SALVATORE,
Research Psychologist
Traffic Systems Division**

Accurate estimation of vehicle speed is necessary for precise psychomotor coordination and anticipation of future response needs. Vehicle control is dependent on information perceived from the complex and changing stimuli of the surrounding environment. The test reported in this article was aimed at determining how the vehicle operator obtained speed estimates from visual stimuli in the field. Response to both foveal and peripheral visual stimulation was recorded to determine which mode of visual stimulation allowed the driver the best information for estimating the speed of the vehicle. The author reports that speed was most accurately estimated through peripheral stimulation. It was also concluded that increased acceleration reduces the accuracy of speed estimation and that deceleration is more effectively sensed than acceleration.

Introduction

FOR THE STUDY reported in this article an apparatus was constructed and experimental procedure developed to test how the vehicle operator obtains speed information from the visual field. In the driving task the driver must process information for directional control or heading, longitudinal control or distance to significant objects, lateral position, and vehicle speed. As driving becomes less and less dependent on muscular strength and more and more a task of exercising control in response to a complex and continuously changing environment, the importance of split-second judgment and psychomotor skill is paramount. The importance of accurately estimating vehicle speed cannot be underestimated because the amount of force exerted by the driver to control the vehicle is dependent on his perception of speed. But more important is the fact that accurate estimation of vehicle speed is required for the driver to achieve smoothness of the psychomotor response. Anticipation of future psychomotor response needs leads to precise psychomotor coordination and short reaction time.

The following list contains definitions for some terms used in this article:

Fovea centralis.—The fovea centralis is a depression at the back of the retina of the eye, the point where the vision is most acute, corresponding to approximately the central 2° of the visual field. In this article the fovea centralis is referred to as fovea.

Interocular distance.—Interocular distance is the horizontal distance between the centers of the pupils of the two eyes when they are in the normal position for distance vision.

Iris diaphragm.—The iris diaphragm is a mechanical device in a camera composed of thin metal leaves arranged and shaped to provide a circular opening that can be varied in size.

Kinesthetic.—Kinesthetic pertains to the sense that yields knowledge of the movements of the body or of its several members.

Parafovea.—The parafovea is the area of of the retina surrounding the fovea, approximately the central 4° of the visual field, excluding the area of the fovea.

Periphery.—Periphery refers to the outward bounds of something, such as vision, as distinguished from its center.

Psychomotor.—Psychomotor refers to the precise coordination of a sensory or ideational process and a motor activity, such as aiming at a target.

Tactual.—Tactual refers to the sense of touch.

Conclusions

From the experiments to determine how the vehicle operator obtains speed information from visual stimuli in the field, the author made several conclusions:

- Speed estimation is more accurate in the periphery than in the fovea.
- The effect of increased acceleration increases all absolute and relative error thus reducing the sensitivity of the visual mode.
- Absolute error is directly proportional to speed and relative error is inversely proportional to speed for the central stimulation only when the influence of acceleration is minimized.

- Under the condition of the experiment, deceleration is more effectively sensed than cceleration.
- Processes underlying speed estimation are far from perfectly understood and the uthor believes a program of research in this area would be fruitful. The dimensions that need to be studied are: exposure time; raveled field structure classified as to width of road or proximity to roadside objects, road markings, and type of pavement; area of etinal stimulation; and speed.

Background

Despite the significance of the operator's ability to estimate speed and the processes underlying it, studies in this area are limited. In laboratory experiments, Poulton (*1*)[1] has reated tracking as an analog to the driving task and has differentiated between course anticipation and speed anticipation. Speed anticipation refers to vehicle control based on the perception of stimulus movement; course anticipation refers to vehicle control based on the perception of course stimulus. Generally laboratory studies use small moving targets usually within the limits of the parafovea. But Gordon in his study has shown that the angular speed is minimal in the fovea centralis and maximum in the periphery for three dimensional movement (*2*).

Field studies of absolute speed estimation have been sparse; most have dealt with speed attainment. In studying speed attainment, Suhr and Lauer (*3*) recorded underestimation at low speeds and overestimation at high-speeds. Barch (*4*), studying speed adaptation, determined that adaptation did not occur for exposure times of up to 8 minutes and that underestimates were made for deceleration. Hakkinen (*5*) compared estimates made by subjects viewing films of the driving situation and estimates made by passengers in an actual traffic situation. Though the split-half reliability for each technique is high, the correlation between techniques was not significantly different from zero.

Basis for Study

In the actual driving situation the rapidity of obtaining a speed estimate is critical. The vehicle operator can make a speed estimate through a foveal or a peripheral view, as shown in figures 1 and 2, respectively. But, the value of the estimate decreases as the time required to make it increases, if the two estimates are of a given accuracy and reliability. One purpose of the research discussed here was to test the hypothesis that speed estimates are more accurate based on peripheral stimulation than foveal stimulation under the condition of equal exposure time. The theoretical discussion of whether the speed of moving objects is perceived directly or by relating perceived spatial displacement to perceived duration was considered. The answer depends partially on the locus of retinal stimulation used in obtaining speed information. In the peripheral view angular speed seems relatively fast and movement is obvious so that recognition of movement of the vehicle through the environment is apprehended directly. In the foveal view movement seems slower so that speed may have to be computed by estimating the distance traversed and the time elasped; this computation procedure is time consuming.

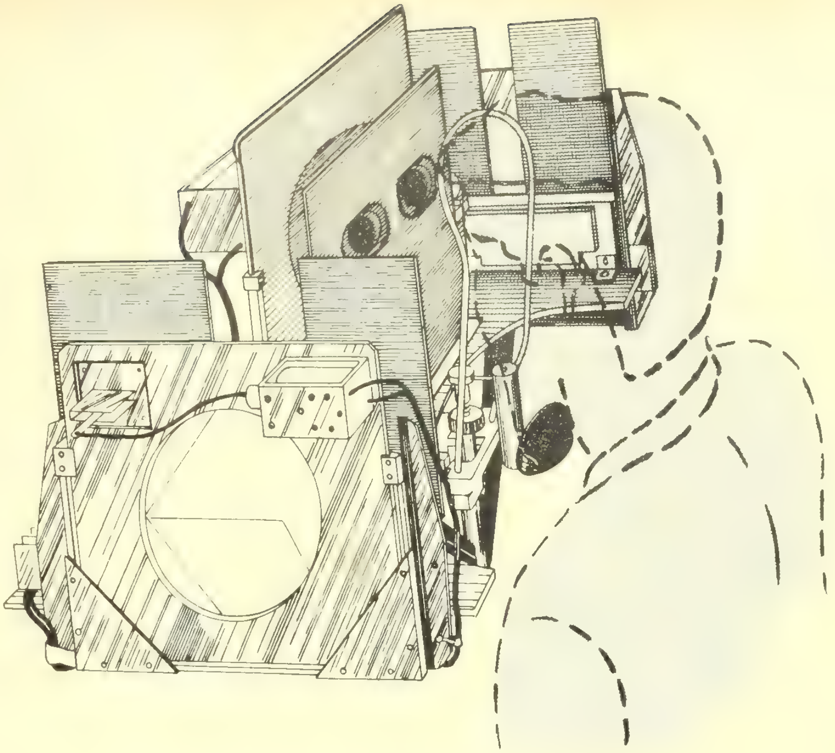

Figure 3.—Test subject and test apparatus.

Two hypotheses were tested in the experiment reported here: (1) estimates of speed based on peripheral stimulation are higher than estimates of speed based on foveal stimulation; (2) estimates of speed based on peripheral stimulation are more accurate than estimates of speed based on foveal stimulation.

Test Apparatus

Experimental apparatus for the speed estimation study was basically two camera iris diaphragms mounted on a base. The purpose of the apparatus was to control the relevant segment of the visual field and time for observation. The distance between iris centers, corresponding to the interocular distance, was adjusted to accommodate individual sight variation. With the eyes of the test subject 2 inches from the iris, the foveal opening could be adjusted in a range from 1° to 30°. The peripheral field was controlled by two slats placed in a groove that ran parallel to the optical axis at a distance of 5 inches on either side of the subject's eye. The rear slat was positioned so that it subtended an angle of 90° at the subject's eye. The forward slat was adjustable so that the amount of the peripheral field available for observation could be varied. The testing apparatus, shown in figures 3 and 4, was constructed with the cooperation of the Engineering Systems Division of the Bureau of Public Roads.

The duration of visibility of the field was controlled by three, 4½-inch diameter shutters. The two shutters were placed on either side of the slats and the third in front of the two irises. An electronic timer controlled the voltage to the solenoids that operated the opening and closing of the shutters. The electronic timer ranged from one-fourth to 64 seconds in the geometric series of nine steps. The subject's head was placed in position by a chin rest attached to the diaphragm base. The base was held in approximate position by a boom held in tension between the floor and the roof of the car and was leveled by two turnbuckles attached to the base from the vehicle visor rods.

Test Subjects

Four subjects were chosen at random from a list of volunteers. The three females and one male in the sample ranged in age from 18 to 37; the mean age was 25 years. Uncorrected vision was tested by a technician on the Bausch and Lomb Ortho-rater; foveal visual acuity was 20/29 or better and the peripheral field was 80° or more on both sides None of the test subjects wore glasses and all had from 2 to 20 years of driving experience, or a mean of 8½ years.

Test Site

The test site was a 5-mile stretch of unopened Interstate 64 a few miles east of Richmond, Va. Although this concrete highway is a 4-lane divided freeway, only the two westbound lanes, each 12 feet wide, were used in the test. Lane and edge striping had not been painted yet so the highway was classified as a semistructured environment. Maximum slope and horizontal curvature were 3 percent and 2° respectively. The longitudinal central joint and the transverse construction joints which were spaced 61.5 feet apart, were filled with approximately 1 inch of conventional black asphalt joint-filler. The right shoulder width was constructed of 10 feet of asphalt paving and 10 feet of earth fill. During the experiment, the vehicle was centered in the right lane. Estimates were never made when nontest vehicles were in the testing area.

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[1] Numbers in parentheses indicate references, which are listed on p. 131.

Experimental Design

The research was carried out as a two-factor design, as shown in figure 5. Factor 2, mode of observation or manner of visual stimulation, was tested for 25° centrally and 25° peripherally. Factor 2, vehicle speed, was tested for 20-, 40-, and 60-m.p.h. speeds. This basic design was used at 1 m.p.h./sec. acceleration and was repeated at 5 m.p.h./sec. acceleration because it was assumed that acceleration would affect the estimates. Four subjects made speed estimates for the 6 experimental conditions, 5 times each, for a total of 120 estimates at each rate of acceleration.

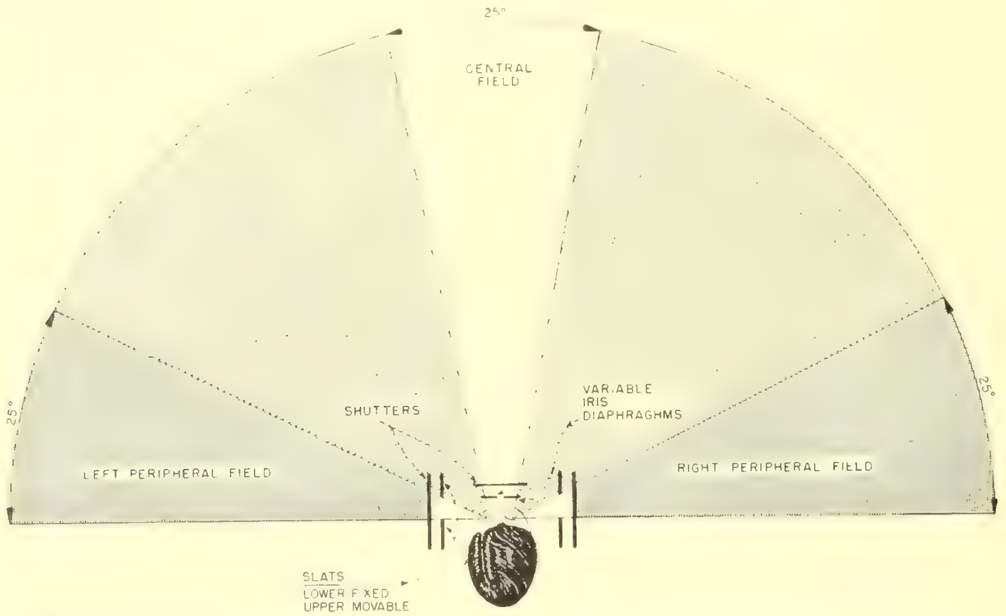

Figure 1.—The foveal and peripheral visual field as seen through the test apparatus.

Two sets of experiments were made. In the first set of experiments a 1 m.p.h./sec. acceleration-deceleration was used to attain the desired actual speed; in the second a 5 m.p.h./sec. acceleration-deceleration was used. Visual exposure time for both experiments was a constant 1 second and a masking noise was piped into the subjects' ears by a headset to eliminate auditory cues. Windows were closed to eliminate wind cues. In set 1, the time between visual exposure to the field was held constant to eliminate the influence of time on estimates. In set 2, visual exposure was made when the actual test speed had been reached. Subjects were instructed to count aloud between visual exposures as a distraction from nonvisual cues of speed. The order of the experimental conditions was semirandom.

Instructions

Test subjects were given the following instructions: The purpose of this experiment is to find out how people estimate speed. Your job in the experiment will be to give verbal estimates of the speed that you think the car is traveling. Give your estimates to the nearest 5 m.p.h., for example, 25, 30, 35 m.p.h., and so on. Your view of the road will be blocked by the apparatus for most of the time that you are in the experiment. Every 45 seconds either the front or side shutters will open for a short time. You will not know ahead of time whether your view will be forward or to the side so be ready to observe in either direction. The best way to accomplish this is to stare straight ahead in what we call the stare mode. Try it. Staring straight ahead you should also be able to see the sides without moving your eyes, as you did in the eye test. A tap on the shoulder from me will indicate that either the front or side shutters will open for a short time and that you should be ready to observe. As soon as the shutter(s) close(s) give your estimate of the speed that you think the car was traveling while the shutter(s) was open. Give your estimate of the speed as soon as the shutter(s) closes. Speed estimates that are delayed cannot be used. If you are not sure of the speed, guess. Do not hesitate. The sequence of events will be: dark phase for 45 seconds; tap on shoulder, be ready to observe; opening of side or front shutter, observe; closing of shutters, report estimated speed to the nearest 5 miles; repetition.

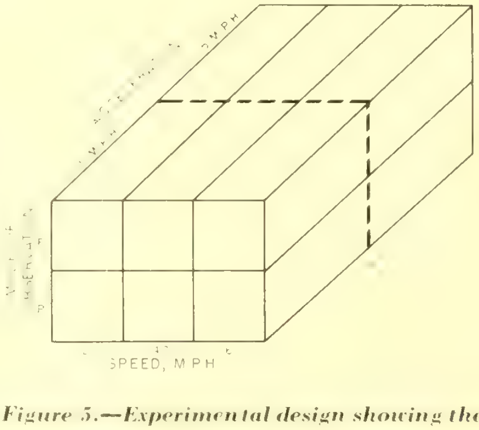

Figure 5.—Experimental design showing the factors of analyses of variance.

Analyses of Variance

Two analyses of variance were made: one for each of the two levels of acceleration. The analyses showed that vehicle speed was extremely significant regardless of the mode of observation or level of acceleration. The mode of observation at the 1 m.p.h./sec. acceleration was significant beyond 0.001—results such as those obtained would occur by chance less than one in a thousand—but, the mode of observation at 5 m.p.h./sec. acceleration was not so significant statistically as it was less than 0.05. Such statistical results show that an interaction was present between acceleration and mode of observation; this interaction is discussed in a subsequent paragraph. However, the lack of interaction between speed and mode of observation at the 1 m.p.h./sec. level of acceleration strengthens the belief that the effectiveness of the perceptual technique is independent of speed when acceleration effects are minimized.

The relation of estimated speed to actual speed for an acceleration of 1 m.p.h./sec. is shown in figure 6. The parameter is the mode of observation. The broken line indicates the ideal speed. Estimates made by the subjects from peripheral stimulation were statistically significantly faster than the estimates they made from foveal stimulation. Thus, hypothesis 1 was confirmed; the foveal estimates were underestimates at all three speeds. The subject estimated that he was moving slower than he actually was. The peripheral speed estimates were slightly less than the actual speed at the 20 and 60 m.p.h. speeds and slightly more at 40 m.p.h. Thus, hypothesis 2, that peripheral stimulation is more conducive to accurate estimates than central stimulation was confirmed. At the 5 m.p.h./sec. acceleration test, subjects shifted their estimates to slower speeds for both peripheral and foveal modes of observation, as shown in figure 7. The peripheral estimates were faster than the foveal estimates at all speeds, but the difference between the two was smaller and not statistically significant.

At the 1 m.p.h./sec. acceleration the complete lack of overlap of both the absolute and the relative errors for the foveal and peripheral stimulation shows the gain in accuracy of speed estimates under the condition of peripheral stimulation. Furthermore, absolute error directly proportional to the speed, and relative error inversely proportional to the speed holds true only for foveal stimulation. For peripheral stimulation no obvious trend in relation of errors

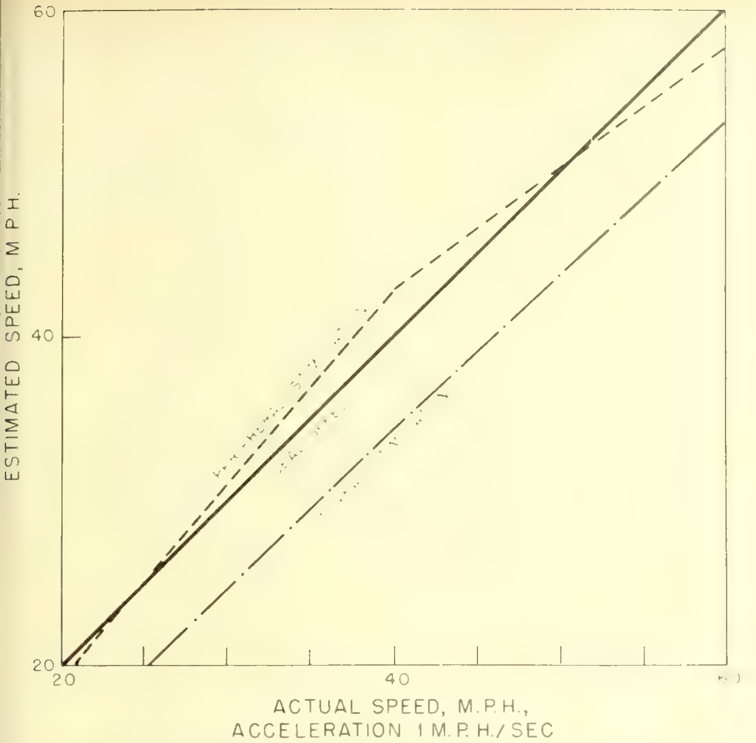

Figure 6.—Estimated speed as a function of actual speed, 1 m.p.h./sec. acceleration.

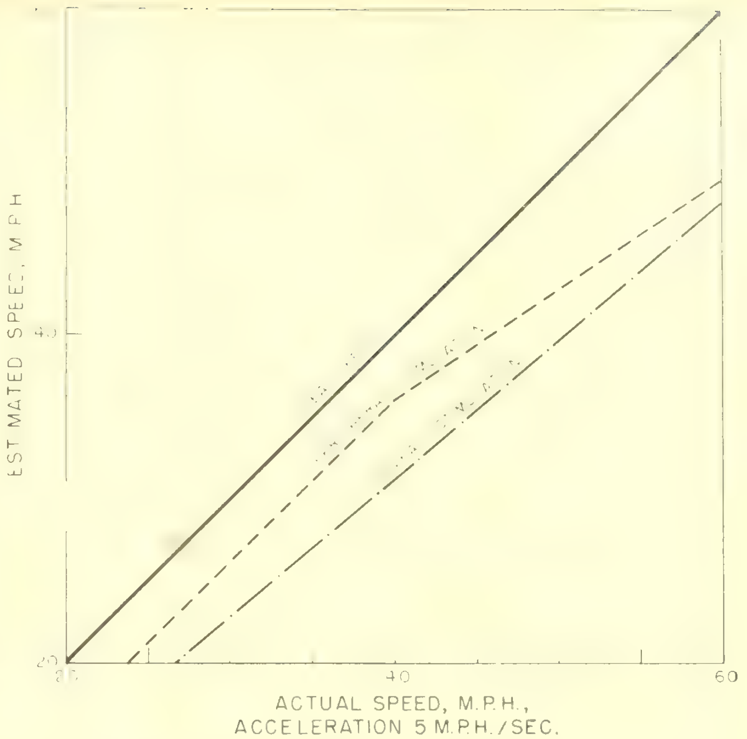

Figure 7.—Estimated speed as a function of actual speed, 5 m.p.h./sec. acceleration.

to speed was noted, and the relative error is best described by a straight line parallel to the ordinate. The general effect of acceleration was an increase in both the absolute and relative errors at each speed for both modes of stimulation. The interaction between acceleration and mode of observation was marked and has the effect of making peripheral and foveal curves look alike. Thus, acceleration tends to cancel out the better sensitivity to the peripheral stimulation. Some difference between peripheral and foveal stimulation was maintained, but for the most part the acceleration cue tended to overpower the visual sensitivity. The net effect was to increase the absolute error making it seem that under the conditions of this experiment, deceleration was more effectively sensed than acceleration.

The possible cues to speed are manifold. The movement of the visual field on the retina, auditory cues of engine and road noise, kinesthetic and tactual cues of acceleration, wind speed, and gas pedal displacement can all play a part when vehicle speed is being estimated. An overall impression incorporating stimulation from the different sources and senses may provide the basis for the best estimation of speed. For the purpose of the study reported here, however, it was assumed that the visual sense was the primary source of speed information and that movement of the visual field is directly proportional to vehicle speed through the environment. It was also assumed that most information necessary to the driving task is obtained visually and, because switching from one sense to another is detrimental to performance under time limitations, it also seemed advantageous to obtain velocity information visually. The discovery of the order and relative weighting of the cues that enter into speed judgments were considered scientifically interesting.

A cursory examination of the relation of the visual field to its external counterpart shows that, for any particular objective speed, the visual field has diverse values of motion depending on the mode of observation. Motion is least in the fovea along the direction of motion. The question arises as to where in the field the observer should concentrate to obtain speed information. The experiment reported here proves that a more accurate assessment of speed information is obtained from peripheral visual stimulation than from foveal visual stimulation. The immediate explanation for this result seems to be that angular speed is much faster in the periphery than in the fovea. For example, at a vehicle speed of 60 m.p.h. and 25° of available visual stimulation, the corresponding maximum angular speed is 1,080° per second peripherally and 81° per second foveally. For the 1-second time constant used, which is generous, the observer was able to respond better to peripheral stimulation. Angular speed in both the foveal and peripheral field is directly proportional to vehicle speed but as the angular speed was faster in the peripheral visual field by more than one order of magnitude, this speed could be scaled more easily. A more stringent test of the relation of the visual field to its external counterpart would be to reduce available observation time toward zero. Such an experiment is planned in which updated equipment will be used.

For the foveal mode of observation at the 1 m.p.h./sec. acceleration, the absolute error was directly proportional to vehicle speed and the relative error was inversely proportional to vehicle speed. Both errors were practically parallel to the ordinate when the stimulation was peripheral. As Hakkinen (*6*) reported a high degree of reliability within the methods of observation when the observer was in motion and the camera in the position of the observer and a zero correlation between modes of observation, the author has assumed that the process underlying the estimates was altered by substitution of foveal for peripheral stimulation. The author also believes that an experienced driver can use this experience in judging speed when he is restricted to foveal mode of observation but that this ability to compensate probably would be eliminated as observation time is reduced toward zero.

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Travel by Motor Vehicles in 1963, 1964, and 1965

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

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Introduction

Motor-vehicle travel during 1965 in the United States has been estimated at 887.6 billion vehicle-miles, an increase of 4.9 percent from 1964. Based on preliminary data, 1966 motor-vehicle travel has been estimated at 932 billion vehicle-miles, a 5-percent increase from 1965. These Public Roads estimates are based on information supplied by State highway departments and toll authorities. Recently, a more accurate interpretation of these data has become possible by use of information collected in 1963 by the Bureau of the Census, and by the State highway departments in cooperation with Public Roads. Accordingly, motor-vehicle travel for 1963 and 1964 has been reestimated as described in the article.

Definitions

Vehicle-miles.—Vehicle-miles refers to the amount of travel by one motor-vehicle traveling 1 mile and includes travel on all highways and streets in the United States.

Registered weight groups.—Registered weight groups are the weights used as a basis for vehicle licensing, separated into intervals according to State registration weight practices.

Trailer combinations.—A trailer combination is a truck or truck tractor pulling one or more trailers and/or a semitrailer.

Motor-fuel consumption.—Motor-fuel consumption is the total consumption of motor fuel by highway vehicles for the year, obtained from State records.

Motor-fuel consumption rate.—Motor-fuel consumption rate is the average rate of motor fuel usage in miles per gallon (m.p.g.).

1965 Travel

The travel by road system and vehicle type changed little from 1964 to 1965, as shown in table 1. In 1965, travel on main rural roads was 37.8 percent of the total vehicle-miles of travel; main rural roads are 14 percent of the Nation's 3.7 million miles of roads and streets. The 1965 travel on urban streets was 47.8 percent of the total travel, urban streets are 14 percent of the total mileage of roads and streets. But only 14.4 percent of the travel in 1965 was on local rural roads although these roads are 72 percent of the total mileage of roads and streets.

Passenger cars in 1965 were 84 percent of the vehicles registered and accounted for 80 percent of the travel. Single-unit trucks accounted for 15 percent of the vehicles registered and 16 percent of the travel; trailer combinations accounted for less than 1 percent of the vehicles registered but 4 percent of the travel. Buses accounted for less than 1 percent of the vehicles registered and the total travel.

Motor-vehicle performance

Average vehicle performance estimates for 1965 were only slightly different than the corresponding estimates for 1964. The average motor-vehicle travel in 1965 was 9,674 miles, almost half of it in cities; and average motor-fuel consumption was 775 gallons at a rate of 12.48 m.p.g. In 1964, the estimated average motor-vehicle travel was 9,698 miles; and average motor-fuel consumption was 778 gallons at the rate of 12.47 m.p.g. Averages for passenger cars in 1965 and 1964 were, respectively: 9,255 and 9,286 miles of travel and 649 and 652 gallons of motor-fuel consumed at a rate of 14.27 and 14.25 m.p.g.

Basis for Estimates

For the first time, it has been possible to provide separate estimates for single-unit trucks and trailer combinations The yearly travel average for trailer combinations, 41,292 miles, is four times the average travel of single-unit trucks. These separate estimates were developed from information obtained in cooperation with the State highway departments from the 1963 special trucking characteristics study (*1*)[1] and from data in the *1963 Census of Transportation.* Both the special study and the Census (*1, 2*) were discussed at the 44th annual meeting of the Highway Research Board.

Adjustment of 1963 Travel Estimates

Based on Census data (*3*), adjusted to reflect travel of government-owned vehicles, the vehicle-miles of 1963 truck travel have been reestimated at 9.65 percent more than previously published by Public Roads in table VM–1. In the reestimate, a computer data file, based on responses for 100,845 individual trucks, was obtained from the Bureau of the Census. Information in the file had been obtained from questionnaires mailed to a probability sample selected from motor-vehicle registration files in each State. The sample was stratified for each State by registered weight groups. Census information identifying the type of vehicle was translated to correspond to descriptions used for truck weight trends, which are based on axle arrangements of single-unit trucks and each type of trailer combination. As expected from such a complex questionnaire, apparently inconsistent information was recorded by some respondents. More than 96 percent of the original records were successfully edited and recoded. A comparison then could be made of the two sets of data. The average vehicle-miles traveled each year was computed for each vehicle type and weight group based only on the questionnaire containing a response to the question on annual travel. This average was expanded to the corresponding number of trucks of each vehicle type and registered weight group in each State in 1963.

Government-owned vehicles

The adjustment of Census data for government-owned vehicles was necessary so that travel by all vehicles would be reflected as it is in the trend data prepared by Public Roads Trend data are based primarily on continuous traffic counts at 2,500 permanent locations and periodic machine and manual traffic counts made by State highway departments. Most counts are made at the roadside, usually on a 6- to 48-hour sampling basis, and all passing vehicles are counted. In the adjustment, single-unit trucks and trailer combinations owned by all levels of government were estimated to have accounted for, respectively, 5.6 percent of the total travel by single-unit trucks and 0.5 percent of travel by trailer combinations.

From evaluation of the new information (*2, 3*), it was concluded that the estimate of vehicle-miles of travel for single-unit trucks reported during the past few years was low. Evidently, estimates based on roadside counts did not fully reflect the substantial increase in the number of pickup trucks with camper bodies used for recreation. At the same time underestimates of urban travel were made because full consideration was not given to the increase in urban travel caused by the introduction of small vans, the increased use of walk-in delivery trucks, and the more extensive use of single-unit trucks in expanding metropolitan areas. Therefore, the 1963 vehicle-miles of travel for single-unit trucks and trailer combinations reported in this article are new estimates based on the 1963 Census data (*2, 3*), adjusted to include estimated travel by government-owned trucks and trailer combinations.

The new estimate for vehicle-miles of travel by single-unit trucks in 1963 was 13.35 percent more than had been estimated from Public Roads trend figures; but for trailer combinations the new estimate was 3.62 percent less than vehicle-miles previously estimated by Public Roads. When the good

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[1] References indicated by italic numbers in parentheses are listed at the end of this article.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1965 and revised data for 1963 and 1964 [1]

Vehicle type	Motor-vehicle travel: Main rural roads	Motor-vehicle travel: Local rural roads	Motor-vehicle travel: Total rural	Motor-vehicle travel: Urban	Motor-vehicle travel: Total	Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption: Total	Motor-fuel consumption: Average per vehicle	Average travel per gallon of fuel consumed
1965										
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gallon*
Passenger cars [2]	254, 975	97, 662	352, 637	356, 663	709, 300	76, 643	9, 255	49, 723	649	14. 27
Buses:										
Commercial	922	184	1, 106	1, 815	2, 921	85. 0	34, 365	628	7, 388	4. 65
School and nonrevenue	687	758	1, 445	318	1, 763	229. 3	7, 689	247	1, 077	7. 14
ALL BUSES	1, 609	942	2, 551	2, 133	4, 684	314. 3	14, 903	875	2, 784	5. 35
All passenger vehicles	256, 584	98, 604	355, 188	358, 796	713, 984	76, 957	9, 278	50, 598	657	14. 11
Cargo vehicles:										
Single-unit trucks	56, 832	28, 378	85, 210	55, 949	141, 159	14, 008	10, 077	13, 848	989	10. 19
Trailer combinations	21, 994	1, 395	23, 389	9, 108	32, 497	787	41, 292	6, 658	8, 460	4. 88
TOTAL	78, 826	29, 773	108, 599	65, 057	173, 656	14, 795	11, 737	20, 506	1, 386	8. 47
ALL MOTOR VEHICLES	335, 410	128, 377	463, 787	423, 853	887, 640	91, 752	9, 674	71, 104	775	12. 48
1964 Revised										
Passenger cars [2]	243, 429	93, 539	336, 968	340, 645	677, 613	72, 969	9, 286	47, 567	652	14. 25
Buses:										
Commercial	908	181	1, 089	1, 803	2, 892	82. 3	35, 140	622	7, 558	4. 65
School and nonrevenue	674	743	1, 417	307	1, 724	223. 1	7, 727	242	1, 085	7. 12
ALL BUSES	1, 582	924	2, 506	2, 110	4, 616	305. 4	15, 115	864	2, 829	5. 34
All passenger vehicles	245, 011	94, 463	339, 474	342, 755	682, 229	73, 274	9, 311	48, 431	661	14. 09
Cargo vehicles:										
Single-unit trucks	52, 929	27, 112	80, 041	53, 670	133, 711	13, 275	10, 072	13, 199	994	10. 13
Trailer combinations	20, 592	1, 307	21, 899	8, 661	30, 560	738	41, 409	6, 271	8, 497	4. 87
TOTAL	73, 521	28, 419	101, 940	62, 331	164, 271	14, 013	11, 723	19, 470	1, 389	8. 43
ALL MOTOR VEHICLES	318, 532	122, 882	441, 414	405, 086	846, 500	87, 287	9, 698	67, 901	778	12. 47
1963 Revised										
Passenger cars [2]	231, 298	89, 080	320, 378	324, 993	645, 371	69, 842	9, 240	45, 246	648	14. 26
Buses:										
Commercial	877	170	1, 047	1, 794	2, 841	82. 2	34, 562	606	7, 372	4. 69
School and nonrevenue	642	708	1, 350	292	1, 642	215. 7	7, 612	232	1, 076	7. 08
ALL BUSES	1, 519	878	2, 397	2, 086	4, 483	297. 9	15, 049	838	2, 813	5. 35
All passenger vehicles	232, 817	89, 958	322, 775	327, 079	649, 854	70, 140	9, 265	46, 084	657	14. 10
Cargo vehicles:										
Single-unit trucks	50, 043	25, 981	76, 024	49, 729	125, 753	12, 654	9, 938	12, 348	976	10. 18
Trailer combinations	19, 900	1, 302	21, 202	8, 614	29, 816	706	42, 232	6, 084	8, 618	4. 90
TOTAL	69, 943	27, 283	97, 226	58, 343	155, 569	13, 360	11, 644	18, 432	1, 380	8. 44
ALL MOTOR VEHICLES	302, 760	117, 241	420, 001	385, 422	805, 423	83, 500	9, 646	64, 516	773	12. 48

[1] For the 50 States and District of Columbia, 1963 data have been adjusted, based in part on the 1963 Census of Transportation Truck Inventory and Use Survey, to provide data separately for single-unit trucks and trailer combinations.

[2] Includes taxicabs and motorcycles: 786,318 in 1963, 984,763 in 1964, and 1,381,956 in 1965, estimated to account for less than 1 percent of all travel: 1963, 0.5 percent; 1964, 0.6 percent; 1965, 0.7 percent, respectively.

agreement in the two sets of data for travel by trailer combinations was noted, an investigation was made for each State of the differences by individual types of combinations. This was possible because most operators of trailer combinations keep fairly complete records. It was hypothesized that an underestimate of travel was more apt to be made for vehicles used infrequently, but, this was not confirmed by a check of other data (*6*). Although proportionally more responses were made by operators of the larger vehicles, when responses were made by operators of smaller vehicles reliable information was given on annual mileage

Motor-Fuel Consumption

Truck motor-fuel consumption estimates for 1963 reported here were based on detailed calculations of motor-fuel consumption rates for gasoline and diesel powered trucks classified by average operating gross weights for 10 types of trucks. These data were combined (table 1) for estimates of motor-fuel consumption for single-unit trucks and trailer combinations. These calculations were based on the 1963 truck weight study reports—an annual State report submitted to Public Roads—and separate estimates of the average registered gross weight of gasoline and diesel powered trucks for each type (*2*). Operating gross weights for gasoline and diesel powered trucks were computed by adjusting the average operating gross weight by the ratio between average registered gross weights of diesel to gasoline powered trucks. In the *Supplementary Report of the Highway Cost Allocation Study* (*4*), figure 12 relates average operating gross weights and type of motor-fuel. The vehicle-miles of travel by truck type were divided by the miles-per-gallon usage rate to obtain the gallons of motor fuel used by all trucks in 1963.

Passenger car fuel consumption

An adjustment also was made in the 1963 estimate for passenger car motor-fuel consumption. The previously published rate of 14.37 m.p.g. has been revised to 14.26 m.p.g. The revised estimate was based on independent analysis of available data and the previously used trend data was not considered. In the reevaluation, annual automobile sales since 1957 were analyzed to identify significant shifts in the proportion of small, medium, and large cars. Also considered was the proportion of cars equipped with automatic transmissions, optional higher powered engines, and power accessories, which affect motor-fuel consumption rates. Data for these rates were taken from an article by Nathan Lieder (*5*) and industry reports. The estimates of motor-fuel consumption by vehicle type were compared and adjusted so the total equaled the total highway use of motor fuel. The 1964 and 1965 motor-fuel consumption for passenger cars was calculated by a similar procedure in which the new 1963 estimate of travel data was used as the base.

Motor-vehicle fuel consumption, 1964 and 1965

Calculations of 1964 and 1965 estimates of motor-fuel consumption were based on data for travel by vehicle type, truck sales by fuel and truck type, and registrations by vehicle type. A sharp increase in the use of heavy (5 or more axles) trailer combinations was noted. The increased average operating gross weights for these heavier combinations ordinarily would reduce the miles per gallon rate; however, the 1964–65 increase in sales of more efficient diesel powered truck tractors tended to offset this expected reduction for trailer combinations. In contrast, the 1964 single-unit truck motor-fuel consumption rate increased from 10.13 m.p.g. in 1963 to 10.19.

In 1965, travel and sales of 4-tired trucks exceeded that for 1964. According to the 1965 truck weight study reports their travel was up 8 percent while sales increased more than 15 percent, according to industry data. The 1965 increase in miles per gallon by single-unit trucks has been attributed primarily to the increase in vehicle-miles of travel by single-unit, 4-tired trucks. The miles per gallon for other single-unit trucks was much lower. The changes discussed and other changes in truck usage were studied to refine the 1964 and 1965 estimates of motor-vehicle fuel consumption for trucks and trailer combinations (table 1).

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Application of Statistical Concepts to Accident Data

BY THE
BUREAU OF PUBLIC ROADS

Reported by [1,2] DONALD A. MORIN,
Urban Transportation Planning Engineer,
Region 8, Portland, Oregon

In this article Morin has pointed out the necessity for applying statistical concepts to traffic accident data. Although procedures for determining the significance of accident data have already been developed little has been accomplished toward their application. Procedures are suggested by which the limited available accident data can be analyzed and erroneous conclusions avoided.

The importance of statistical analysis of data is illustrated by one State's failure to identify unstable accident rates. In this State's program, high priorities were assigned to some highway sections that had few accidents and low priorities were assigned to other sections that had numerous accidents. This and other examples of inaccurate interpretation of accident data illustrate the need for applying statistical concepts to accident data to determine the significance of accident rates. The author also discusses the normal chance variation in accident rates, the minimum accident rates that definitely exceed an established tolerable rate, and the percentage of accident reduction needed to establish the reliability of a road section improvement.

Introduction

THE CONCERN being expressed nationwide about the highway accident toll has generated a flood of magazine articles, promoted legislation, and encouraged discussion within technical circles as to what can and should be done. No pat answer or agreement on the solution has been reached. Some believe the solutions hinge on stricter enforcement of traffic regulations, others on more and better driver education, others on improved highways, and others on safer vehicles. Agreement seems to exist on one point, however, and that is the lack of adequate accident records to enable agencies to establish accurate conclusions on highway accidents. A method for obtaining adequate records is not proposed in this article, but methods are suggested by which better analyses of the limited available accident data can be made so that erroneous conclusions will not be made.

Conclusions

From the investigation of existing statistical procedures for interpreting available accident data several conclusions have been drawn.

- Although statistical procedures based on quality control concepts were developed and successfully applied to accident data a number of years ago, traffic and highway engineers have made only limited use of them to date.
- A procedure must be found to accelerate the use of existing statistical procedures over the pace of the past to permit concentration of manpower and resources in those areas that offer the best prospects for improvement.
- When applied, statistical controls have enabled engineers to make valid conclusions as to the effectiveness of safety improvement projects.
- Application of statistical procedures has also enabled engineers to determine the amount of variation inherent in accident rate data and thereby has minimized the possibility of erroneous conclusions.

Statistical Concepts Needed

Accidents are scarce events—a few accidents per million vehicle-miles of travel; the universe in which accidents occur is extremely large—many hundreds of millions of vehicle-miles. As a statistician might say, "We are dealing with a small sample of a large population." In this situation, erroneous conclusions can easily be made unless the engineer uses well established statistical concepts to determine the significance of the accident data. Although most highway and traffic engineers are not specifically trained in statistics, they should be able to recognize when the services of statisticians are needed. Examples cited in the following paragraphs illustrate situations that required the application of statistical concepts to the analysis of accident data.

In one State, priorities on hazardous rural and urban highway sections were established by ranking the sections according to their annual accident rate, that is, the first priority was given to the section that had the highest annual accident rate. The fallacy of this procedure is evident from the assignment of priority 4 to a 1-mile section of a rural road on which only 3 accidents had occurred, none of which had caused personal injury or fatality, and the assignment of priority 47 to another 1-mile section of rural road on which 186 accidents had caused 89 personal injuries. In this same State, a 0.2-mile section of urban road on which 66 accidents had caused 12 injuries was ranked 13th, and another 0.2-mile section of road on which 123 accidents had caused 70 injuries and 3 fatalities was ranked 190th. Of course, sections with few accidents had high accident rates and vice versa because of the wide variation in the number of vehicle-miles of travel on the sections. Road sections of equal length were not differentiated in terms of their respective vehicle-miles of travel.

In another State, the annual State highway accident report listed, by route, the accident rate for each control section. When submitted, each control section that had a rate of 10 or more accidents per million vehicle-miles of travel was underlined in red. Examination of the accident data for these sections showed that many of the sections having a rate of 10 or more were not significant nor worthy of closer scrutiny because only one or two accidents had occurred; the few accidents coupled with the low figures for vehicle-miles of travel produced high accident rates. Other seemingly significant sections on which the vehicle-miles of travel were large had not been singled out by the red underlining because the accident rates were slightly less than 10.

In another instance, highway and traffic engineers determined that the large number of accidents at a complicated intersection could be curtailed by the installation of overhead sign bridges, improved signals, and some limited approach widening. During the 3-month period after completion of the improvement project, the number of accidents had decreased by eight. A press release, *Intersection Made Safer*, stated that the city's ". . . most dangerous intersection . . . apparently has been tamed." At the end of the following 8-month period, however, there had been only 5 fewer accidents, a 12-percent decrease that was far short of the 38-percent decrease needed to assure reliability. No press release or publicity was given to this.

The proper application of statistical concepts could make it possible to avoid pitfalls, like the ones cited, by determining the amount of normal chance variation that should be expected in accident rates, the minimum accident rate that definitely exceeds an established tolerable rate, and the percent of accident reduction needed to establish the reliability of a road improvement project.

An attempt is made in the rest of this article to summarize statistical concepts that have been developed and applied to this problem area. Although many of these applications were developed more than 10

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[1] Presented at the 46th annual meeting of the Highway Research Board, Washington, D.C., January 1967.

[2] Mr. Morin is now Chief, Public Transportation Branch, Urban Planning Division, Office of Planning, Washington, D.C.

years ago, only limited use has been made of them.

Control Limits

The Office of Technical Services of the U.S. Department of Commerce in 1958 distributed a manual (*1*)[3] that described a procedure for determining the amount of variation in the accident rate that could be expected from chance probability for any highway control section. The input required is the overall accident rate for the highway and the number of vehicle-miles of travel on the control sections. However, in 1966 S. K. Dietz discovered an error in the equations as originally presented in the Technical Service manual and HRB Bulletins 117 (*2*) and 341 (*3*). The validity was improved by omitting the correction term, 0.829/m. The authors of the original equations have concurred in Mr. Dietz's correction. By applying the corrected equations, both upper and lower control limits on the overall accident rate can be established for each control section.

$$\text{Upper control limit}=\lambda+2.576\sqrt{\lambda/m}+\frac{1}{2m} \qquad (1)$$

$$\text{Lower control limit}=\lambda-2.576\sqrt{\lambda/m}-\frac{1}{2m} \qquad (2)$$

Where,

λ = Overall accident rate for the highway.
m = Vehicle-miles of travel on a control section.

Equations (1) and (2) provide expressions for the upper control limit that has a probability 1−P of being equalled or exceeded by chance. To define the upper control limit as one that has a probability 1−P of being exceeded, the equation would be,

$$\text{Upper control limit}=\lambda+2.576\sqrt{\lambda/m}-\frac{1}{2m} \qquad (3)$$

This is identical to equation (1) except that the sign of the $\frac{1}{2m}$ term is reversed.

To define the lower control limit as one that has a probability 1−P of being exceeded by a more negative number the equation would be,

$$\text{Lower control limit}=\lambda-2.576\sqrt{\lambda/m}+\frac{1}{2m} \qquad (4)$$

This is identical to equation (2) except that the sign of the $\frac{1}{2m}$ term is reversed.

Chance Variation

It is possible with the equations cited to compare the actual accident rate for each highway section with the control limits on the overall accident rate to determine whether the variation from the overall rate is more than could be attributed to chance. The coefficient of the second term in the equation, 2.576, assumes a 0.995 or ½ of 1 percent probability level that either the upper or lower control limit could be exceeded by chance variation in the observed accident rate, or a 0.990 or 1-percent probability level that both the upper and lower control limits could be exceeded by chance variation. For example, a value of 2.576 would mean that for 1 percent of the road sections the accident rate could be expected to fall beyond the control limits by chance even though there is nothing out of the ordinary about them, and that ½ of 1 percent of the accident rates for the road sections could be expected to fall above the upper or below the lower control limit by chance even though there is nothing out of the ordinary about them.

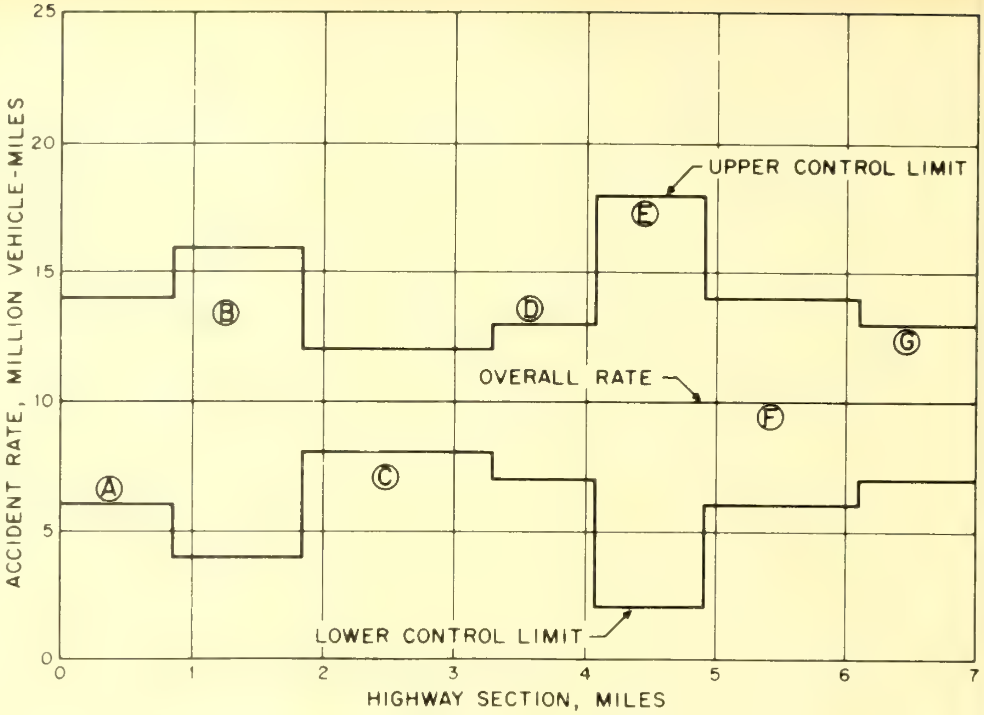

Figure 1.—Relation of actual accident rates to control limits on overall accident rates.

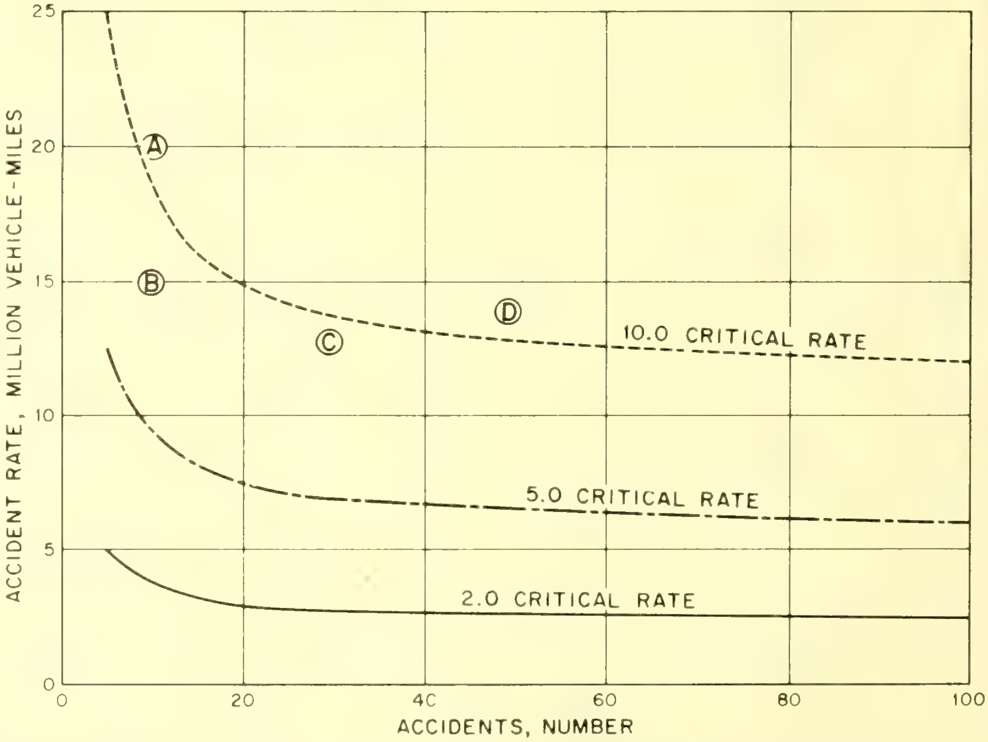

Figure 2.—Minimum accident rates statistically significantly higher than critical rate.

Other coefficients that would change the probability of labeling a rate as out of the ordinary when it is in fact normal could be used: 1.960 for 5 percent false detection of both or 2½ percent of either; 1.645 for 10 percent false detection of both or 5 percent of either; 1.440 for 15 percent false detection of both or 7½ percent of either; 1.282 for 20 percent false detection of both or 10 percent of either.

A comparison of the actual accident rates to control limits on the overall accident rate

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[3] Italic numbers in parentheses indicate the references listed on p. 137.

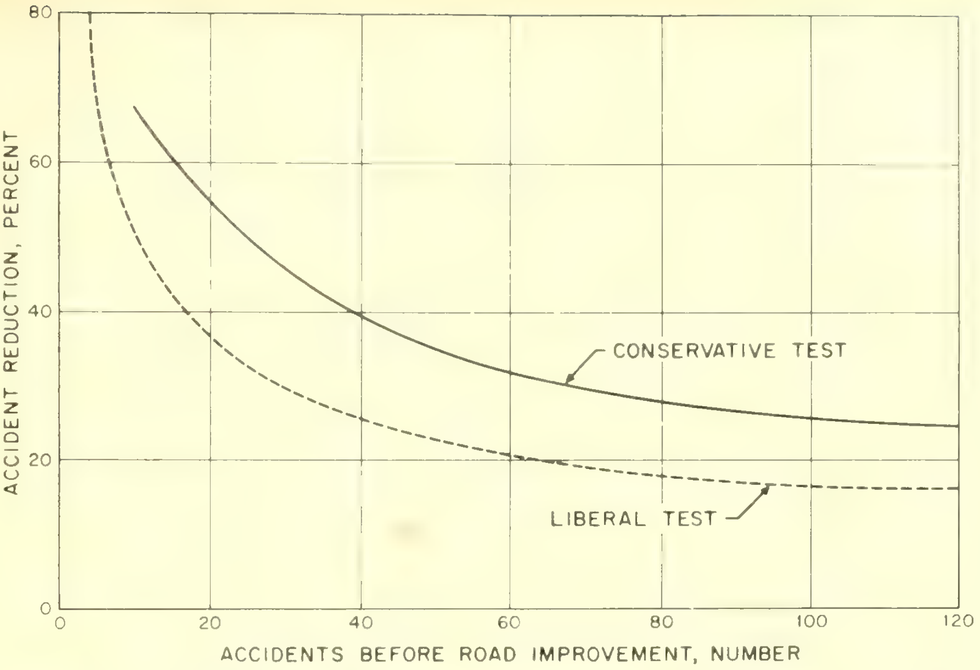

Figure 3.—Curves for use in determining statistical significance of reduction in number of accidents.

can be made from figure 1. In the figure the observed accident rate at point E seems high. Because of the few vehicle-miles of travel on this section, the control limits differed widely from the overall rate. It should be concluded that the apparently high rate at point E was not worthy of investigation as it was within the range of variation that could be expected by chance. By comparison, the observed accident rate at point D does not appear to be very much higher than the overall rate and is much lower than the rate at point E. But, because point D is outside the control limit for that section, its variation from the overall rate is more than could be attributed to chance. Accidents on this section of the road, therefore, warrant analysis. The accident rate indicated by point C should also be investigated, as something other than chance variation caused the accident rate to be so much lower than the overall rate. Investigation of the causes of this low rate might provide a means for attaining similar low rates on other road sections.

Rudy (*3*), in 1962 described the application in Connecticut of the procedure for determining the amount of variation in the accident rate that could be expected from chance probability. The Montana State Highway Commission recently programed this procedure for their IBM 1620 computer and successfully ran their 1965 accident data. The program printed out the upper and lower control limits and the observed accident rate for each highway control section. The results were indicated in the 1965 Annual Accident Report by an asterisk alongside the computed accident rate for sections where these rates were outside the control limits. Because these variations from the overall rate did not occur by chance, the next logical step is to find the reasons for the abnormally high and low accident rates.

Under consideration is a procedure to assemble in one report all possible data concerning time and exact location of accidents, accident type, weather conditions, roadway alinement and cross section details, sight distance, and so on for those road sections where accident rates seem to be out of control. A team consisting of a traffic engineer, design engineer, maintenance engineer, and law enforcement officer would study the assembled data and physically look at the highway sections to determine the reason for the abnormal accident rates.

Critical Rate Analysis

A somewhat simpler application of statistical analyses to accident rate data, using the same basic concepts, is being used in Idaho. The procedure requires that the highway authority establish the critical accident rate, that is, the highest accident rate the authority agrees to tolerate. Then, for any established critical rate, a minimum accident rate can be determined for any number of accidents that is statistically significantly higher than the critical rate of accidents on a particular road section. Minimum accident rates that exceed the established critical rates are illustrated in table 1 and figure 2.

In figure 2, the accident rates at points A through D and the established critical rate of 10 accidents per million vehicle-miles of travel indicate that the rate of 15 based on 10 accidents—point B—and 13 based on 30 accidents—point C—are not significantly higher than the critical rate of 10. The accident rates shown by points A and D, however, should be attributed to something other than chance variation; the rates of 20 based on 10 accidents—point A—and 14 based on 50 accidents—point D—are statistically significantly higher than the critical rate of 10 accidents per million vehicle-miles. Although the Idaho procedure provides a quick method for identifying road sections that have rates significantly higher than a predetermined critical or tolerable rate, it does not pinpoint road sections that have significantly low accident rates.

Effectiveness of Improvement Projects

In 1959 Dr. R. M. Michaels published a procedure for determining the statistical significance of the percentage of reduction in the number of accidents on a road section after improvement as compared to the number before improvement (*4*). The Poisson distribution was considered an appropriate approximation of the accident probability for the liberal test. The chi-square test was used to determine whether there was a significant difference between the before and after samples for the conservative test. One of the main advantages of this procedure is that the engineer can test for significance knowing only the number of accidents before the road improvement and the percentage of accident reduction after improvement of the road section. The test involves spotting the percentage of reduction on a graph, as shown in figure 3, to determine the significance of the reduction. If the point falls below the conservative line, for example, the reduction is not significant by the conservative test.

Table 1.—Minimum accident rate statistically significantly larger than critical accident rate (probability level=0.95)

Critical accident rate per million vehicle-miles is —	Minimum accident rate per million vehicle-miles when Actual number of accidents per road section is 5	10	20	30	50	100
	Minimum rate	*Minimum rate*	*Minimum rate*	*Minimum rate*	*Minimum rate*	*Minimum rate*
2.0	5.0	3.7	3.0	2.8	2.6	2.4
3.0	7.5	5.6	4.5	4.2	3.9	3.6
5.0	12.5	9.3	7.5	7.0	6.4	5.9
7.0	17.5	13.0	10.5	9.8	9.0	8.3
10.0	25.0	18.5	15.0	14.0	12.8	11.9

The failure of engineers to apply the statistical procedures can lead to false conclusions about highway improvement. An engineer in a recent article (*5*) reported that no definite conclusions could be drawn from a safety experiment because data were too limited and not statistically significant. However, application of Dr. Michael's procedure showed that the percentage of reduction in accidents for the safety experiment was statistically significant by the conservative test. Another article (*6*) reported the success of a safety project; but statistical analysis of the percentage of reduction of accidents showed that the accidents had not been reduced significantly.

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(*1*) *Manual for the Application of Statistical Techniques for Use in Accident Control*, by Jack W. Dunlap, Jesse Orlansky, and Herbert H. Jacobs, U.S. Department of Commerce,

continued on p. 150

Economic Study of Luminaire Mounting Heights for Highway Lighting Systems

BY THE OFFICE OF
ENGINEERING AND OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by [1] JAMES A. THOMPSON, Chief,
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and BENDER I. FANSLER, Electrical Engineer,
Highway Standards and Design Division

An economic study is a valuable prerequisite for an engineering project. The authors of the study reported in this article believe that an economic evaluation of the different highway lighting systems would prove helpful to highway engineers in designing new systems. Details of an economic and engineering study of lighting designs for different mounting heights of luminaries for use on controlled-access highways and how the mounting heights affect lighting costs are presented here. Also, formulas are given for making evaluations for the different mounting heights of luminaries.

From the research it was concluded that an economic study, such as that reported in this article, would provide more economical and effective lighting system designs and ultimately contribute to improved highway safety and esthetics. Based on the study reported here, the previously accepted standard, 30-foot mounting heights for luminaries, may be considered undesirable; the authors believe that highway lighting systems designed to use luminaires mounted at heights of 40 to 50 feet would be preferable in terms of economy, effectiveness, safety, esthetics, and flexibility for future modification.

Introduction

A WISER expenditure of public funds for highway lighting in relation to design should be possible from the information presented in this article. Details of an economic and engineering study of lighting designs for different mounting heights of luminaires for use on controlled-access highways and how the mounting heights affect lighting costs are presented. The method developed for evaluating different lighting designs should be useful in the design of more economical highway lighting systems. Although not a prime factor of the research study reported here, lighting system design can affect the esthetic properties of a highway and adequate lighting may increase safety on highways at night. Neither the degree of safety provided by night lighting nor the possible hazard of lighting poles along a highway has been formally studied, but the fewest possible poles per mile is a logical design consideration in relation to highway safety.

One of the factors sparking the study reported here is the increase in attention being given lighting of controlled-access highways in urban areas. As traffic volumes and operating speeds of vehicles have increased, a general public awareness of the need for and a demand for highway lighting has developed. Although several highway agencies have extensive lighting programs, many have limited programs or none at all. Despite the fact that each year highway engineers are considering the installation of more lighting systems, resistance to their cost plagues decision makers.

Although an economic study generally is a basic requisite for an engineering project, highway agencies heretofore have made little use of economic studies when designing highway lighting systems. The information and techniques discussed in this article should be helpful for evaluating proposed lighting projects. It is believed the evaluation methods discussed here emphasize the benefits to be derived from use of economic studies in relation to design. Methods for evaluating some of the cost differences of alternative designs are given, and other information is given on factors that may contribute to the design choice—factors that are impracticable to evaluate monetarily, such as esthetics and safety. An economic study should support planning and decision making so that more efficient and economical highway lighting installations can be provided—systems that will contribute to the safety and comfort of the road-user.

The authors caution that the cost information presented in this article represents relative values used only for examples in the economic evaluation procedures. The costs cited should not be used for project justification nor for budget preparations.

Conclusions and Findings

Application of the information and techniques for evaluating highway lighting system designs given in this article should be helpful to highway engineers responsible for designing new systems, according to the conclusions made by the authors. The evaluation of present design techniques by the cost-effectiveness procedure discussed in this article is a basis that can be used for evaluating different designs. Use of this economic evaluation during planning of a lighting system could assure a wiser expenditure of public funds for more effective and esthetic lighting systems. These improved lighting systems could also improve safety conditions on the highways. Specific findings and related conclusions based on the research are given in the following statements.

On the basis of the research reported here, the previously accepted standard of 30-foot mounting heights for luminaires may be considered undesirable for divided highways. Highway lighting systems designed to use luminaires mounted at heights of 40 to 50 feet would be more economical and effective than the designs for luminaries mounted at 30 feet. Use of the higher mounting heights in lighting system designs generally would provide safer and more esthetic lighting. The designs using the higher mounting heights are more flexible and can be readily modified to use new lamp and luminaire improvements. Recent trends in lamp development are toward increased lamp efficiency and larger lumen output.

The uniformity of illumination should be studied and analyzed because the maximum to minimum ratio of illumination uniformity seems to be a more logical basis for comparison of the lighting system's effectiveness than the average to minimum ratio currently in use.

The authors also have concluded that continuous fluorescent bridge rail lighting, mounted at a low height, is not a wise investment of public funds because installation cost is high for such a system, questions abound as to the effectiveness of bridge rail lighting, and maintenance difficulties are many.

Definite conclusions regarding towerlighting for interchange areas generally cannot be based on the evaluation of a single interchange. Design engineers should make a cost-effectivness study for each individual interchange for which a lighting system is being planned because many geometric designs are available for the different requirements at each interchange.

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[1] Presented at the 46th annual meeting of the Highway Research Board Washington, D.C., Jan. 1967.

The annual cost of towerlighting or floodlighting may be approximately the same as the annual cost of a design for 30-foot mounting heights. The initial cost for each of the three tower designs evaluated in the research reported here was less than the initial cost for a 30-foot mounting height design.

Study Methods

In the research reported here, only one direction of a highway was used to develop an evaluation method for determining the most suitable and economical highway lighting system. Geometric and lighting design criteria were based on current design standards and principles, and they were selected so that the principal variable would be the mounted height of the luminaire. The cost and designs for the most commonly used mounting height of 30 feet were compared to the same factors for mounting heights of luminaires at 40, 45, and 50 feet; on bridge rail lighting at a height of 3½ feet; and floodlights (a type of luminaire) mounted at a height of 100 feet in towers or on poles. Costs were computed for the following listed designs: (1) 250-watt lamps on 2-lane roadways; (2) 400-watt lamps on 2-, 3-, and 4-lane roadways; (3) 700-watt lamps on 3- and 4-lane roadways; (4) 1,000-watt lamps on 4-lane roadways; (5) bridge rail lighting; and (6) interchange area floodlighting.

Only designs for divided, controlled-access highways were considered. Comparable designs for lighting systems were compared for only one direction of a roadway for 2-, 3-, and 4-lane pavements. All lanes were 12 feet wide and the right shoulders were 10 feet wide. The luminaires were located over the right edge of the traveled way. Bridge rail lighting systems were evaluated with the roadway lighting systems but lighting for interchange areas was evaluated separately.

A design level of average initial horizontal illumination of 1.0 footcandle and an average to minimum uniformity ratio that did not exceed 3 to 1 was used for all overhead lighting system evaluations. A few design adjustments were made to obtain an acceptable lighting uniformity ratio, which caused some deviation from the average 1.0 footcandle of initial horizontal illumination. The minimum acceptable level of average initial illumination was established as 0.8 footcandle.

All overhead lighting designs considered were based on one manufacturer's design charts for use of clear mercury lamps. At the higher mounting heights, increased lamp wattages were required to maintain the 1.0 footcandle initial illumination at a fairly constant level. Therefore, 700-watt (34,600-lumen) and 1,000-watt (53,000-lumen) mercury lamps were used when design requirements exceeded the capacity of the 400-watt (19,500-lumen) lamp. The 250-watt (10,500-lumen) mercury lamps were used only for designs for 30-foot mounting heights. For bridge rail lighting design, 42-inch, 33-watt (2,190 lumens at 300 milliamperes) fluorescent lamps in 6-foot luminaires were studied.

For bridge rail lighting, in which light poles would be eliminated, fluorescent lights are mounted in a continuous line adjacent to or in lieu of a bridge railing. Although the concept and design for low-mounted lighting is different from overhead lighting, comparisons were made for installations that were judged to be comparable. Horizontal footcandle illumination, glare, and uniformity of illumination, which are the most common performance criteria used in designing a lighting system, did not seem to be a logical basis for comparing the low- and overhead-mounted lights. Other researchers have noted that a different method should be used to compute the average value of roadway illumination than is used to evaluate overhead lighting. Although the designs are different, for the purposes of this study the low-mounted lighting designs were considered to be similar to the overhead lighting system designs.

Other design criteria assumed to be constant for the lighting systems so that the principal variable would be the mounting height were: (1) Galvanized steel poles, anchor base, and concrete foundations; (2) 12-foot brackets, luminaire located over the edge of traveled way; (3) underground wiring system using cable-conduit; (4) multiple system circuitry; (5) power delivered at secondary voltage, no load center considered; (6) median wide enough so that lighting from opposite lanes would not be a factor; (7) comparable characteristics of pavement reflection so adjustments would not be required in computing average initial horizontal illumination; (8) time controls equivalent for all systems; (9) medium, semicutoff luminaires of IES types II and III; and (10) ballast in luminaires.

Interchange floodlighting systems may be designed so that mounting heights of the luminaires range from 80 to 150 feet. Each interchange should be evaluated separately to determine the mounting heights that will best fit the geometric features. A floodlighting system differs somewhat in concept from the 30- to 50-foot mounting height designs for roadway lighting systems. Footcandles computed for controlled lens lighting may be similar to footcandles computed for floodlighting but roadway brightness measured by footlamberts may be different. The floodlighting design generally used was considered to be comparable to the overhead system designs used in the study reported in this article. A sketch of the interchange area used in the study discussed herein is shown in figure 1.

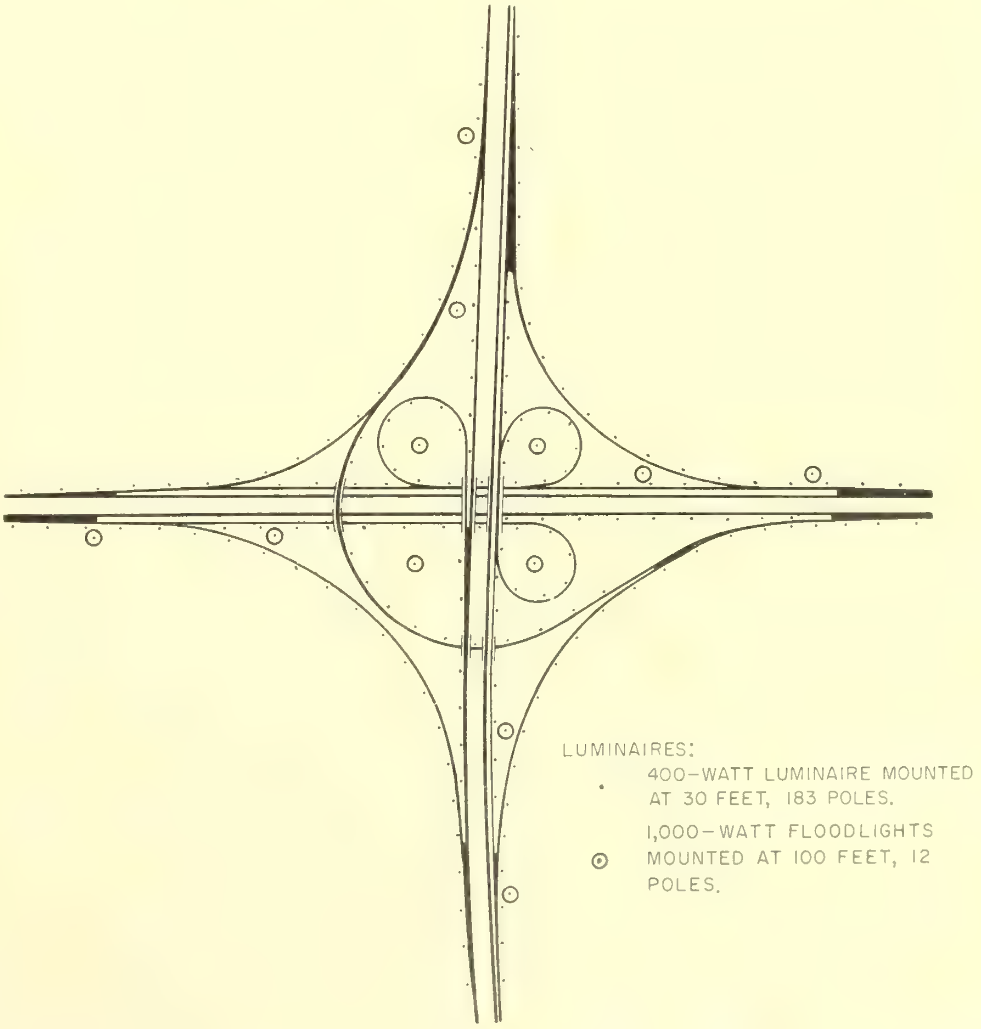
LUMINAIRES:
• 400-WATT LUMINAIRE MOUNTED AT 30 FEET, 183 POLES.
⊙ 1,000-WATT FLOODLIGHTS MOUNTED AT 100 FEET, 12 POLES.

Figure 1.—Interchange layout of lighting poles for 30-foot and 100-foot mounting heights of luminaires.

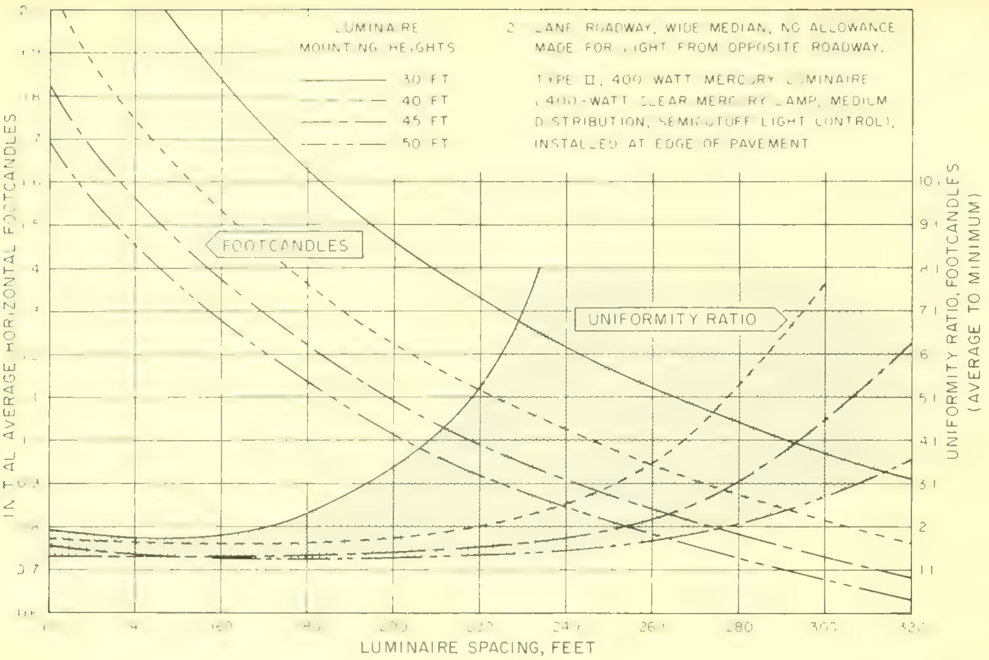

Figure 2.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 2-lane roadway.

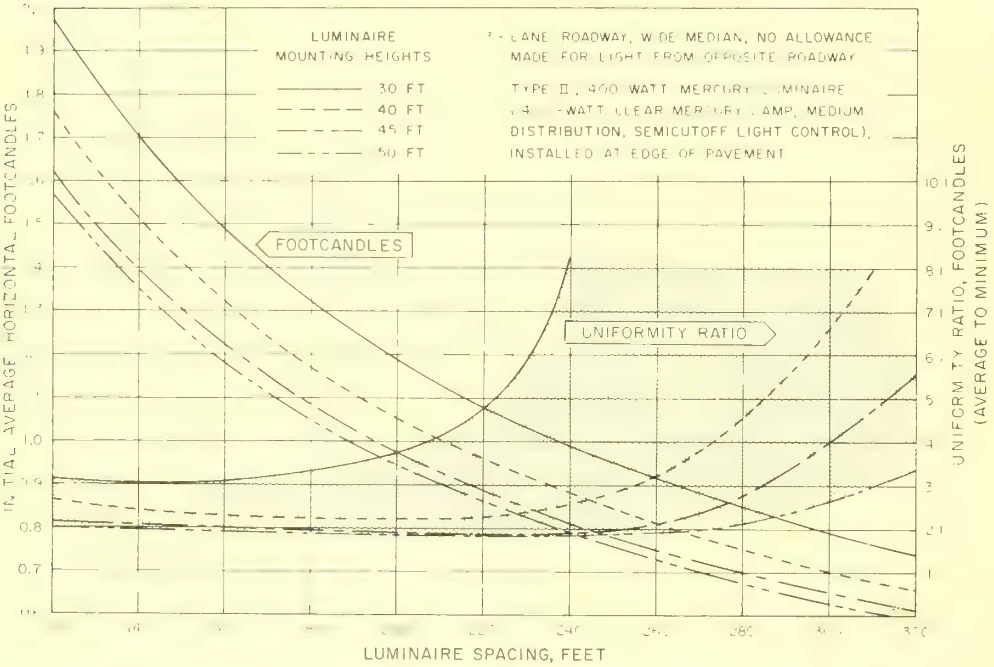

Figure 3.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 3-lane roadway.

Table 1.—Luminaire cleaning and lamp replacement costs

Luminaires			Lamps		
Mounting height	Cleaning schedule	Estimated cost of cleaning one	Wattage	Group replacement schedule	Estimated cost
Feet			*Watts*	*Years*	
3½	Semiannually	[1]$2.00	33	2	$2.00
30	Semiannually	1.50	250 400	4	8.00 8.00
40	Annually	1.50	400 700 1,000	4	8.00 14.00 16.00
45	Annually	1.75	400 700 1,000	4	8.00 14.00 16.00
50	Annually	2.00	400 700 1,000	4	8.00 14.00 16.00
100	Biannually	3.00	400 1,000	4	8.00 16.00

[1] Cleaning costs were based on current maintenance practice but more frequent cleaning obviously would be required to keep maintained illumination from this type of fixture comparable to overhead lighting.

The area was 6.75 miles of separate roadways. Floodlighting designs using 400-watt and 1,000-watt lamps were evaluated. Also an industrial type, symmetrical distribution luminaire design, in which 1,000-watt lamps are used, also was evaluated.

Cost Data Conditions and Estimates

The cost data in this article are based on information considered typical of national averages. These data are given as a basis for determining relative initial, operating, and maintenance costs for lighting systems in which the luminaires are installed at different mounting heights. These cost data should not be used as a guide for estimating the cost of specific highway lighting projects because material, delivery charges, electric energy, labor rates, and other costs may vary at different geographical locations. Initial costs for individual items were combined to obtain a total initial cost per mile, which was statistically converted and is restated as an equivalent annual cost. Luminaire maintenance and lamp replacement costs also were computed and are stated as equivalent annual costs. The estimated costs of luminaire cleaning and lamp replacement were based on maintenance being performed by owners and users. Repairs necessary because of vandalism, pole knockdowns, and other miscellaneous factors were not considered in the evaluation.

The basic formula used to determine the equivalent annual capital cost, *EAC*, of a lighting system for a life expectancy of n years from an initial cost, C, at an interest rate of i percent is,

$$EAC=C\ \frac{i(1+i)^n}{(1+i)^n-1} \qquad (1)$$

Where,

$n=20$ years life expectancy for lighting systems evaluated in study reported here.

$i=6$ percent interest.

$\frac{i(1+i)^n}{(1+i)^n}=$ Uniform series capital recovery factor, *crf*, at an interest rate of 6 percent and life expectancy of 20 years.

Therefore, for the computations discussed in this article,

$$EAC=C\cdot crf \qquad (1a)$$

The basic formula used to determine the present worth, *PW*, of a single investment, *I*, n years in the future at an interest rate of i percent is,

$$PW \text{ of } I=I\ \frac{1}{(1+i)^n} \qquad (2)$$

Where,

$\frac{1}{(1+i)^n}=$ Single payment present worth factor, pwf'.

The formula for determining present worth therefore becomes,

$$PW \text{ of } I=I\cdot pwf' \qquad (2a)$$

Normally, the lighting system constructed and operated by a government agency is financed from road-user taxes in a method similar to that used to finance roadway construction. The road-user taxpayer, if allowed to keep it, could invest this tax money and earn a return. This lost-

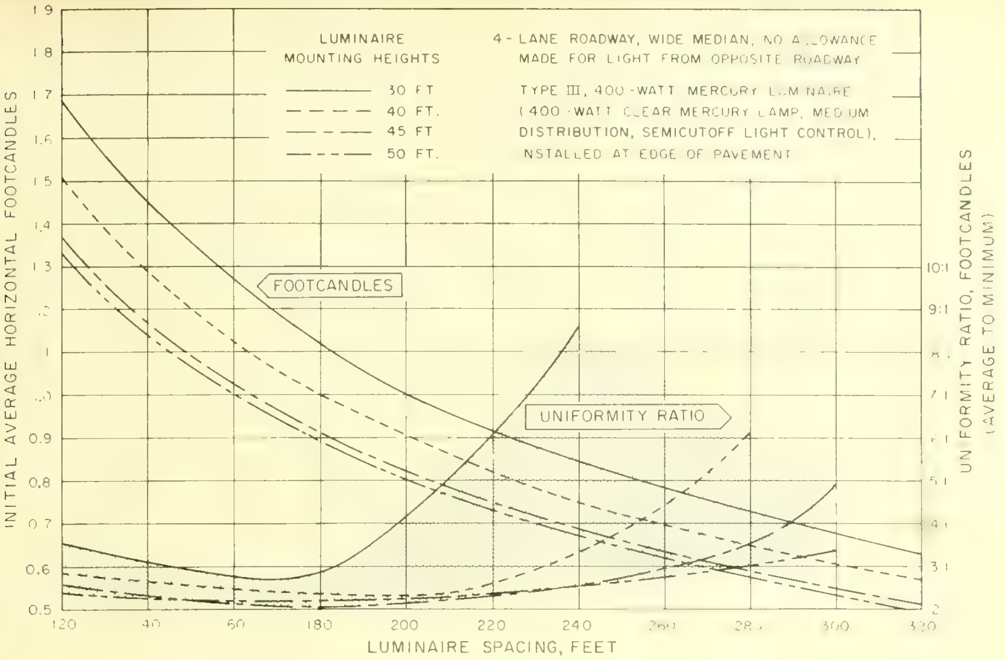

Figure 4.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 4-lane roadway.

Table 2.—Material and installation cost estimates for different luminaire mounting heights

Item	Luminaire mounting heights, feet—					
	$3\frac{1}{2}$	30	40	45	50	100
	Dollars	*Dollars*	*Dollars*	*Dollars*	*Dollars*	*Dollars*
Luminaire and ballast:						
6-foot fluorescent	126	-	---	--	-	
250-w. mercury	-	92	-			-
400-w. mercury	- -	92	92	92	92	-
700-w. mercury	- -	---	144	144	144	-
1,000-w. mercury		---	158	158	158	-
400-w. mercury floodlight	-	---		-	-	125
1,000-w. floodlight	---	-	-	-	-	200
Lamps						
42-inch, T6 fluorescent	2	--		-	-	--
250 w. mercury		8		-	-	---
400-w. mercury		8	8	8	8	8
700-w. mercury		-	14	14	14	-
1,000-w. mercury	---	---	16	16	16	16
Poles	--	200	250	275	325	2,000
Installation per luminaire [1]	40	350	400	425	450	[2] 750

[1] Includes cost for foundation, bolts, wiring, conduit, trenching, and all miscellaneous labor and materials.

[2] Per pole.

investment-opportunity cost should be the minimum interest figure used for determinations of the equivalent annual cost for an initial investment in a highway lighting system. A minimum interest rate should be established that is based on rates of investment opportunities foregone by the taxpayers, but it should be tempered by the element of risk for the 20-year predicted life of the lighting system.

The minimum attractive interest rate should include a safety factor as recognition that even the best engineering estimates are subject to error. Therefore, an interest rate of 6 percent has been used for present worth and capital recovery computations. A 20-year equipment life and no salvage value was used because it was estimated that 20 years is the economic life of most of the system components. It was assumed that the lighting system would be owned by a government agency, which would eliminate tax and insurance costs from the evaluation.

Procedures for maintaining a highway lighting system should always be considered in the design of the system. However, because of the infinite variations in mounting heights and the uncertainty in determining a maintenance factor for bridge rail lighting, and to a lesser degree for the 100-foot mounting heights, maintenance factors such as lumen maintenance and dirt were not included in the evaluation of the highway designs in the study reported here. The omission of maintenance factors permitted logical comparisons of the designs.

Luminaire cleaning schedules vary; they depend on the mounting height, highway geometrics, traffic volumes, and location. The luminaire and lamp maintenance cost data used in the study reported in this article are shown in table 1. Material and installation cost estimates are shown in table 2. Cost summary data are given in tables 3 through 6. The total kilowatt electric load per luminaire was based on lamp wattage, plus ballast loss wattage, plus a line loss of 5 percent. The lighting operation was estimated at 4,000 hours per year and the assumed current cost at $0.015 per kilowatt-hour.

Safety

Few subjects have received so much attention and so little opposition as highway safety. The three major variables of highway safety are the driver, the vehicle, and the highway. Each variable considered separately is complex and indefinite, but combined, the variables present a mass of intangibles so nebulous and replete with insupportable opinions that it is impracticable to establish costs for accidents.

Information in some recent study reports indicates that lighting contributed to safer highway operations during darkness. But, formal research has not been reported that evaluates the degree of safety provided at night by highway lighting. Neither has the degree of hazard created by lighting poles along the highways been established for either day or night. Regardless of the lack of conclusive evidence, it seems logical, when considering highway safety, to favor a lighting system designed for the fewest possible number of poles per mile. It also seems logical to assume that operation in an interchange area would be safer if the number of lighting poles were reduced and located farther from the edge of the travelway. Towerlighting in lieu of roadside poles would provide such a situation. In addition, lighting the entire interchange area rather than just the roadways might improve safety.

The ability to see an object is reduced by glare in the field of view. The glare may be reduced by increasing the luminaire mounting height when the candlepower values remain constant. If glare is reduced, it follows that the result will be better visibility and improved safety. The authors, supported by observations, believe that an improvement in the uniformity of illumination, even when a slight reduction in level of illumination is necessary, would provide better highway lighting. This is one of the advantages to be gained from higher mounting heights and should be evaluated as a safety improvement.

Bridge rail lighting cannot be evaluated in the same manner as general highway lighting; poles on bridges are not considered hazards because thay are located on top or behind the bridge parapet. Because the mounting height of the bridge rail lighting is approximately on a level with the driver's eyes, any resultant glare would be a negative value in highway safety consideration. Also, because of the lack of light directed to the top and rear of the vehicles, a negative safety value may be introduced in the evaluation of bridge rail lighting. But driving conditions during fog or other bad weather may be improved by bridge rail lighting because roadway delineation is improved.

Esthetics

All other design features considered equal, the height of the lighting pole can either enhance or detract from the esthetic quality of the highway. On narrow roadways, lighting from 30-foot mounting heights of luminaires is satisfactory. On wide roadways designed with a wide median or more than one median, a 30-foot mounting height may require four or more rows of poles. The result could be an unsightly forest of poles. A higher mounting height, combined when necessary either with 700-watt or 1,000-watt luminaires, sometimes may permit a reduction in both the number of rows and number of poles and may improve the appearance of the highway at all times.

The taller poles are esthetically acceptable when the ratios of roadway widths to pole heights are considered. When the design norm is considered to be 24-foot, 2-lane, 2-way roadways (two, 12-foot lanes and two, 10-foot shoulders) equipped with 30-foot poles, the ratio of roadway width to height of pole is 44 to 30, or approximately 1.5 to 1. Therefore, a 50-foot pole should be acceptable for roadway widths of 75 feet or more, and 100-foot poles or towers should be acceptable for roadway widths of 150 feet or more. Also, in areas where the type of property development adjacent to the highway is higher than the lighting poles—such as industrial plants, high rise apartment buildings, and so on, or where deep roadway cuts exist—the taller poles or lighting towers blend more readily with the local environment than the shorter poles. But, if the adjacent area has one-story dwellings and the roadway cuts are shallow, use of a shorter pole may be more desirable.

Towerlighting in wide interchange areas seems to be esthetically desirable where acceptable width to height ratios exist. Lighting at night of landscaped areas between ramps enhances the appearance of the entire interchange area. The spill of light off the highway, which may occur when the mounting heights are higher than the surrounding areas, could be either a positive or a negative factor in the design evaluation of highway lighting. The quality of the factor would depend upon the adjacent property and the property owner. In a highly developed area where crime is a problem, spilled light could be an asset for owners of business and residential property. In relation to police protection, lighting is an asset in any area. Spilled light could be a negative factor and a source of complaints in private residential areas where crime is not a problem. In apartment dwelling and business areas, lighting is normally furnished in walking and parking areas, so spilled light from the highway may be desirable.

Table 3.—Cost summary for 2-lane roadway

	Luminaire mounting heights, feet—					
	30	30	40	45	45	50
DESIGN DATA, 2-LANE ROADWAY						
Luminaire:						
Type	II	II	II	II	II	II
Spacing feet	190	195	250	280	220	210
Per mile number	28	27	21	19	24	25
Lamp watts	250	400	400	400	400	400
Illumination:						
Avg. to minimum ratio	3 to 1	3 to 1	3 to 1	3 to 1	1.6 to 1	1.4 to 1
Avg. initial horizontal footcandles	0.83	1.50	1.00	0.79	1.00	1.00
Minimum do	0.29	0.50	0.33	0.27	0.64	0.72
COST DATA, 2-LANE ROADWAY						
Initial cost per mile	$18,200	$17,550	$15,750	$15,200	$19,200	$21,875
Annual cost:						
Equivalent capital do	1,587	1,530	1,373	1,325	1,674	1,907
Equivalent maintenance do	129	124	65	64	81	90
Power	512	770	599	542	684	713
TOTAL	2,228	2,424	2,037	1,931	2,439	2,710

Table 4.—Cost summary for 3-lane roadway

	Luminaire mounting heights, feet—							
	30	40	40	40	45	45	50	50
DESIGN DATA, 3-LANE ROADWAY								
Luminaire:								
Type	II	II	II	II	II	II	II	II
Spacing feet	150	255	220	225	220	265	190	290
Per mile number	35	21	24	24	24	20	28	18
Lamp watts	400	400	400	700	400	700	400	700
Illumination:								
Avg. to minimum ratio	3 to 1	3 to 1	2.3 to 1	3 to 1	1.8 to 1	3 to 1	1.8 to 1	3 to 1
Avg. initial horizontal footcandles	1.60	0.83	0.96	1.29	0.90	1.02	1.00	0.85
Minimum do	0.53	0.27	0.42	0.44	0.50	0.33	0.55	0.28
COST DATA, 3-LANE ROADWAY								
Initial cost per mile	$22,750	$15,750	$18,000	$19,392	$19,200	$17,160	$24,500	$16,794
Annual cost:								
Equivalent capital do	1,983	1,373	1,569	1,691	1,674	1,496	2,136	1,464
Equivalent maintenance do	161	65	75	104	81	91	101	87
Power	998	599	684	1,174	684	978	798	880
TOTAL	3,142	2,037	2,328	2,969	2,439	2,565	3,035	2,431

Table 5.—Cost summary for 4-lane roadway

	Luminaire mounting heights, feet—								
	30	40	40	40	45	45	50	50	50
DESIGN DATA, 4-LANE ROADWAY									
Luminaire:									
Type	III	III	II	III	II	III	III	II	III
Spacing feet	185	180	220	210	255	250	160	280	265
Per mile number	29	29	24	25	21	21	33	19	20
Lamp watts	400	400	700	1,000	700	1,000	400	700	1,000
Illumination:									
Average to minimum ratio	3 to 1	2.4 to 1	3 to 1	3 to 1	3 to 1	3 to 1	2.2 to 1	3 to 1	3 to 1
Average initial horizontal footcandles	1.09	1.00	1.13	1.80	0.91	1.42	1.00	0.80	1.28
Minimum do	0.36	0.43	0.36	0.60	0.30	0.47	0.47	0.27	0.43
COST DATA, 4-LANE ROADWAY									
Initial cost per mile	$18,850	$21,750	$19,392	$20,600	$18,018	$18,354	$28,875	$17,727	$18,980
Annual cost:									
Equivalent capital do	1,643	1,896	1,691	1,796	1,571	1,600	2,517	1,545	1,655
Equivalent maintenance do	134	90	104	118	96	104	119	92	104
Power	827	827	1,174	1,725	1,027	1,449	941	929	1,380
TOTAL	2,604	2,813	2,969	3,639	2,694	3,153	3,577	2,566	3,139

Bridge rail lighting has been promoted as an esthetic improvement even though it may break the continuity of overhead lighting for the highway. The use of rail lighting rather than lighting poles on bridges seems to present a more pleasing appearance during the day. This factor would weigh more heavily in design for bridge lighting on a parkway or scenic highway. As in the evaluation of safety, a dollar value cannot be given to the esthetic qualities of highway lighting, but for specific conditions, some weight should be given to choosing a lighting design that would fit or blend with the highway and adjacent property.

Results of Study

The safest and most esthetic overhead lighting system may be considered the one that provides adequate and effective illumination with the fewest number of poles. The number of poles per mile can be reduced by use of higher mounting heights combined, when necessary, with more wattage for luminaires and lamps. As poles are the most costly item of the lighting system, a system design that reduces the number of poles generally will offset the cost of taller poles, larger foundations, and larger luminaires. Lamps are a small part of the cost.

Three factors are of prime consideration in the effectiveness of any highway lighting system: the level of illumination, the uniformity of illumination, and the control of glare. The uniformity of illumination may be more important than the footcandle level of illumination. And, as the mounting height of lamps is increased, the apparent improvement in light distribution may be better than a comparison of average to minimum illumination uniformity ratios would indicate. It seems that the ratio of maximum to minimum illumination factors should receive more consideration when alternatives are evaluated. Additional study to determine a more positive evaluation of light distribution related to uniformity in level of illumination is recommended by the authors. Results of such a study might show that the road-users ability to perform driving tasks may be improved more by better uniformity in level of illumination than by an increase in the footcandles of illumination.

Luminaires having cutoff vertical light distribution will help to reduce glare and may provide better visibility for the motorists than the semicutoff luminaires. The least glare control is possible when noncutoff luminaires are used, and their use probably should not be considered for expressway lighting. When medians are narrow, a high mounting height improves distribution of light on the opposite roadway. Because the position of the luminaire in relation to the traveled way is not as critical, many times it may be possible to save on the initial cost of a system by use of shorter bracket arms. When the 30-foot mounting heights are used, a pronounced bright spot is present under or near each luminaire. The size and brightness contrast of these pools of light can be reduced considerably by use of higher mountings—less variation is present in pavement brightness and the frequency of eye adaptation is lessened because the driver is not traveling through a succession of intermittent bright spots.

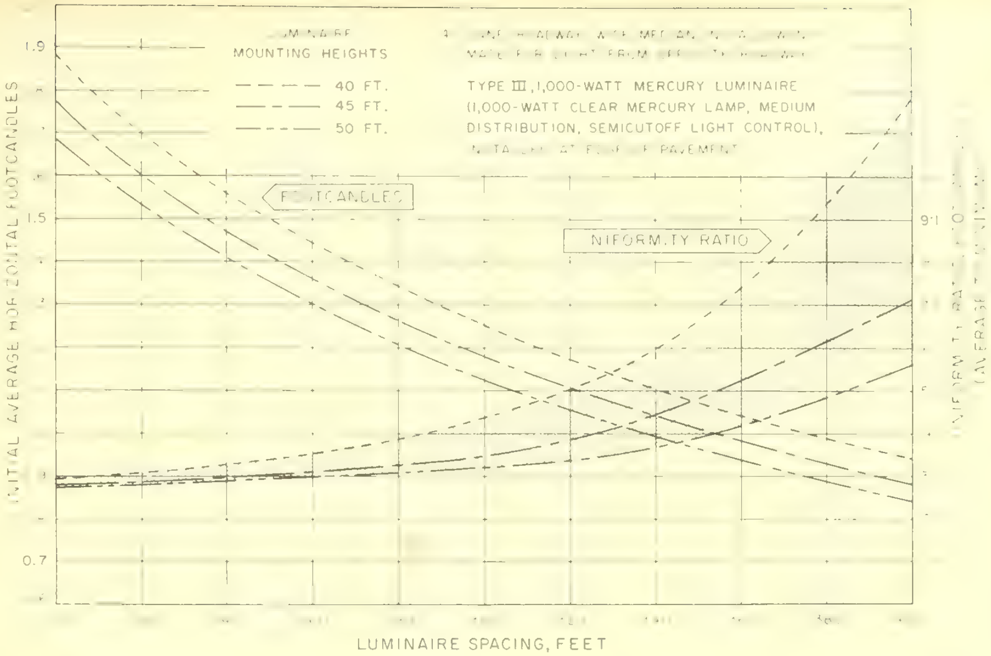

Figure 5.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 4-lane roadway.

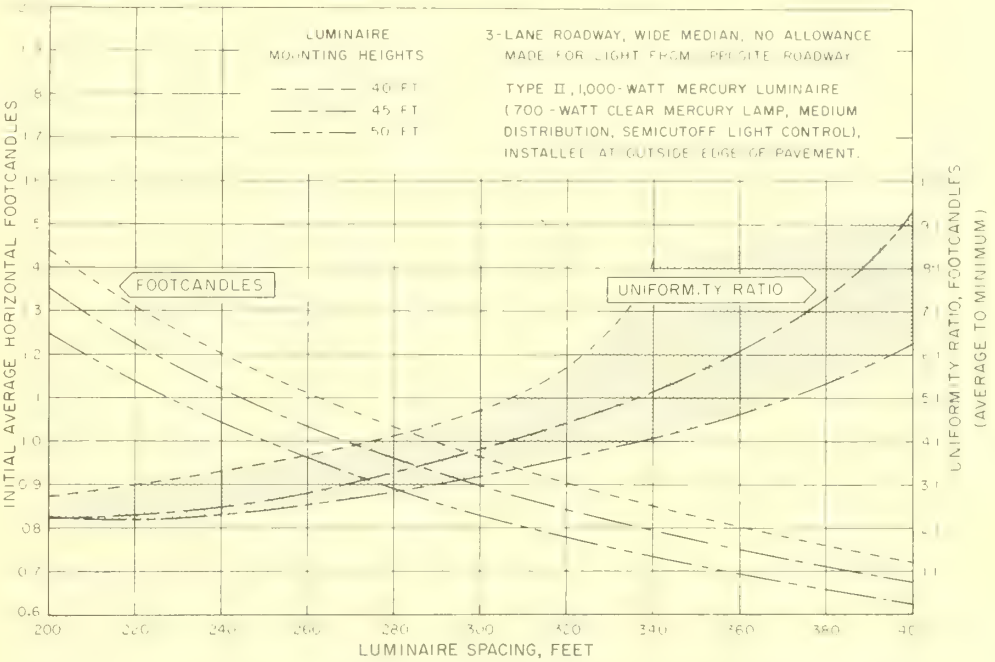

Figure 6.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 3-lane roadway.

According to results of a cost-effective analysis, a 30-foot mounting height for luminaires seldom should be used for lighting a divided, controlled-access highway. The curves in figures 2 through 7, which show the relation of luminaire spacing to lighting uniformity, illustrate that less change occurs in lighting uniformity in relation to an increase in luminaire spacing if the mounting height is increased. This relation also indicates that differences that may exist between calculated and actual measured uniformity would be less as the mounting height is increased.

Data that can be obtained from curves shown in figures 2 through 7 can be an aid in the preliminary design of a lighting system. For example, figure 2 shows that for a 30-foot mounting height of luminaires on a 2-lane roadway, 1.5 initial average horizontal footcandles of illumination are required to obtain a 3 to 1 lighting uniformity ratio and 1.4 footcandles for a 4 to 1 uniformity ratio. But at the 40-foot mounting height of luminaires, 0.8 to 1.2 footcandles of initial average horizontal level of illumination can be obtained when uniformity ratios are about 8 to 1 or 1.7

Table 6.—Cost summary for interchange floodlighting [1]

		Luminaire mounting heights, feet—			
		30	100	100	100
DESIGN DATA, INTERCHANGE FLOODLIGHTING					
Light distribution	type	II	Flood	Flood	V
Poles	number	183	12	12	27
Luminaires	number	183	492	204	108
Per pole	number	1	41	17	4
Lamp	watts	400	400	1,000	1,000
Illumination					
Average to minimum	ratio	3 to 1	Approx. 3 to 1	Approx. 3 to 1	Approx. 2 to 1
Average initial horizontal	footcandles	1.5	Approx. 1.0	Approx. 1.0	Approx. 1.0
COST DATA, INTERCHANGE FLOODLIGHTING					
Initial cost		$118,950	$98,436	$77,064	$101,628
Annual cost:					
Equivalent capital		10,370	8,582	6,718	8,860
Equivalent maintenance		844	2,268	1,269	672
Power		5,216	14,022	14,076	7,452
TOTAL		16,430	24,872	22,063	16,984

[1] Through roadways are two, 12-foot lanes and all ramps have 1 lane except the directional ramp, which has 2 lanes.

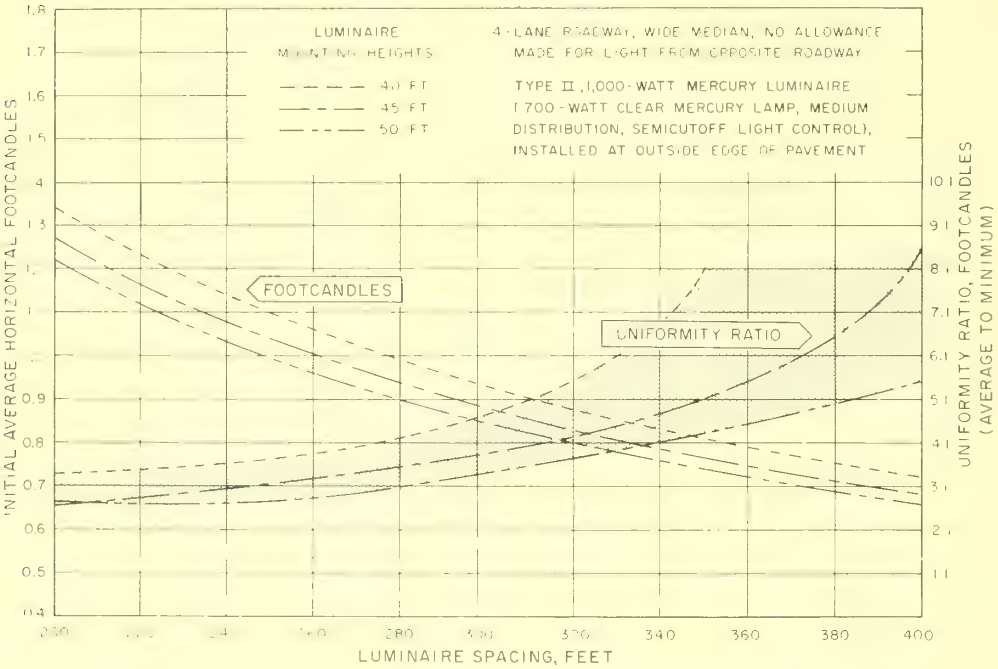

Figure 7.—Initial average footcandles and lighting uniformity ratio for different mounting heights and luminaire spacing for 4-lane roadway.

Table 7.—Interest factors used in cost computations of designs for lighting systems

Years	6-percent compound interest factors [1]	
	Single payment present worth	Uniform series capital recovery
1	0.9434	1.06000
2	0.8900	0.54544
3	0.8396	0.37411
4	0.7921	0.28859
5	0.7473	0.23740
6	0.7050	0.20336
7	0.6651	0.17914
8	0.6274	0.16104
9	0.5919	0.14702
10	0.5584	0.13587
11	0.5268	0.12679
12	0.4970	0.11928
13	0.4688	0.11296
14	0.4423	0.10758
15	0.4173	0.10296
16	0.3936	0.09895
17	0.3714	0.09544
18	0.3503	0.09236
19	0.3305	0.08962
20	0.3118	0.08718

[1] The 6-percent compound interest factors for single payment present worth and uniform series capital recovery calculations were based on investments made at the end of each year (maintenance, replacement, and operation costs were assumed to be charges paid at the end of each year). Zero time ($n=0$) was assumed to be the day the installation was completed and operational.

to 1. To obtain the same level of illumination, the uniformity ratio varies from 2.7 to 1 to 1.4 to 1 for a 45-foot mounting height, and from 1.6 to 1 to 1.2 to 1 for a 50-foot mounting height.

An analysis of the design and cost data tables shows that a 45-foot mounting height would be the most economical lighting design for a 2-lane roadway. The 45-foot mounting height also would be better than a 30-foot mounting height in relation to safety and esthetics. But a 50-foot mounting height would provide the most effective lighting, the best uniformity in illumination, and the least glare. The 30-foot mounting height would provide the most initial average horizontal footcandles.

Analysis of data for a 3-lane roadway, given in table 4, shows that a mounting height of 40 feet would be the most economical design. A 45-foot mounting height would provide the most effective lighting. At a mounting height of 50 feet, glare would be least; also, the 50-foot mounting height design would provide the best highway lighting system in relation to safety and esthetics. On the basis of the cost-effectiveness evaluation, the use of either a 45- or 50-foot design mounting height would be favored. On a 4-lane roadway, as shown in table 5, the 50-foot mounting height design would be a better lighting system on the basis of economy, uniformity, effectiveness, glare, safety, and esthetics. The 40-foot mounting height design would produce the most initial footcandles of illumination.

Bridge rail lighting, continuous rows of low mounted fluorescent luminaires placed adjacent to or in lieu of a bridge railing, eliminates the need for lighting poles and overhead luminaires. This type of lighting should be restricted to locations where overhead lighting cannot be used. The total annual cost for such an installation is approximately 10 times that of conventional overhead systems. Pavement brightness requirements may be met on 2-lane roadways at the 3½-foot mounting height, but whether these requirements are met on 3- and 4-lane roadways is questionable. Although a rail lighting system contributes to the esthetic appearance of a bridge and helps delineate the roadway at night, the problems inherent in maintaining a low mounted fluorescent system, coupled with the increase in annual cost, generally should rule out consideration of such a design. Exposure of luminaires to dirt from frequent splashing of moisture on the highway makes it impracticable to maintain the same degree of cleanliness for luminaires mounted at 3½ feet as for overhead lighting. Use of bridge rail lighting designs should be reserved for special situations.

The estimated initial cost for a lighting system using 6-foot fluorescent luminaires is $220,200 per mile of highway when 1,625 luminaires, mounted at 3½ feet, are installed in two rows. The equivalent annual capital cost per mile would be $19,197; equivalent annual maintenance would be $7,995; and annual power cost would be $4,290.

Flexibility in choice of equipment and design of highway lighting systems seems to increase in relation to the mounting height of the systems. Recent lamp developments could encourage the use of towerlighting or floodlighting in the future. Interchange floodlighting has been used in Europe, but no installations have been made in this country.

(continued on p. 150)

Severance Case Studies—Bridging the Gap Between Findings and Their Application

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by[1] FLOYD I. THIEL, Economist,
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Amounts being received by owners of land taken for highway right-of-way construction for the National System of Interstate and Defense Highways and other Federal-aid highways, as payments for land taken, damages, and sales, often total more than had been anticipated, according to findings from studies of severance effects. Receipts often have totaled more than the estimated values, and losses have been relatively few and small.

The disparities in the estimated values and the actual values shown by analyses of findings—from the case studies in the Public Roads severance effects bank and 2,262 narrative reports on remainder sales—emphasize the savings that can be made. The findings also indicate that underpayments to landowners sometimes are being made.

The importance of applying the available findings from previous case studies to new acquisitions is stressed in this article. Attention is called to the possible source(s) of information, especially for current acquisitions that involve takings for unusual types of properties such as churches, golf courses, and/or schools. By applying these findings, highway takings can be evaluated more closely so that right-of-way can be acquired economically and owners can be paid equably.

Findings from the Public Roads severance bank of cases have been evaluated according to the benefits or losses to landowners, and findings from 2,262 remainder sales have been analyzed on an aggregate basis for different situations. Some of the facets of the analyses pertained to the relation of estimated values of remainder sales to usage before and after severance, and whether the remainder was landlocked or located near an interchange.

Introduction

THE GAP between the availability and application of findings from severance studies—land economic studies—for acquisition of land for highway right-of-way is of concern to all highway officials. Information on the type of findings available from severance studies and suggestions as to how these findings may be of use to appraisers and others concerned with highway construction are discussed in this article. Two sources of information are analyzed: the Bureau of Public Roads bank of severance studies, which is a central file of information; and reports on 2,262 remainder sales listed in *Narrative Reports of Highway Severance Effects: Index and Summary Analysis* (hereafter referred to as the Index), distributed to Public Roads and State highway department personnel.

The gap between findings and their application has occurred primarily because appraisers and others involved in right-of-way takings sometimes do not have access to the pertinent information when they need it. Although information has been collected on approximately 5,000 sales of highway severed property, difficulties seem to arise in getting the right information to the right place at the right time.

Beginning in 1961, the Bureau of Public Roads began collecting information on the effect of taking part of a property for highway right-of-way in a severance effects bank, which now includes 3,000 cases covering 4,000 sales of remainders from these cases. In addition, narrative reports have been collected on more than 2,262 remainder sales. Approximately 40 percent of the severance cases covered by the narrative reports are now recorded in the Public Roads severance bank maintained on electronic data processing equipment. The findings from these studies were intended to be the information that would be used to determine the present and future monetary effects of partial takings of property for right-of-way. These narrative reports have been useful for other purposes such as public relations and land use planning.

Although gaps exist between findings and application of the findings, some success has been achieved in using information from these studies. For example, landlocked areas had commonly been estimated to be damaged 90 percent by severance. But severance studies have shown that such damages usually amount to only about 10 percent of the original value of the land; this finding is now being applied to right-of-way settlements. Thus, some savings in acquisition costs of right-of-way have been accomplished by more accurate estimates of the value of landlocked parcels, as shown in figure 1.

The case study approach is basic for an understanding of the effects of right-of-way acquisition and for providing information in a form that can be applied to future land acquisition appraisals. As the States have completed more studies, the size of the task of recording and filing the data so that the information can be obtained readily has become larger. To ease this task, some States and Public Roads are relying on electronic data processing equipment. When a computer is used to record data, case information can be sorted and analyzed more readily and findings on comparable case situations can be provided more quickly and easily.

Public Roads has had limited success in using the computer to locate information on comparable studies. Some requests, however, have been filled for comparable cases that could be used in current land acquisition situations. Sometimes searches of the bank have been unsuccessful because time required for programs having higher work priorities prevented an adequate search. Also, the Public Roads severance bank contains only a few of the unusual types of cases.

Obtaining findings for the severance bank on takings from properties used for special purposes is very desirable; such information would increase the effectiveness of the data bank. Some of these special purpose property uses are for roller coasters, golf courses, riding stables, schools, and churches. To make it easier to enter data in the severance bank on cases pertaining to special use properties, a shortened case study form has been developed by Public Roads; this form is compatible to the longer form now being used and to the computer programs that have been developed.

Conclusions

Bridging the gap between findings from land economic studies and the application of the findings depend primarily on organization of the findings and making them available in usable form. Case studies for comparable situations can ordinarily be obtained from

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[1] Presented to the Land Economic Study Conference at the 12th annual National Seminar of American Right-of-Way Association, Denver, Colo., June 7, 1966.

[2] Ruth B. Ross, Gwen Van Domelen, John Yasnowsky, Jr., and M. Donald Nolden, also contributed work on which this article was based.

State highway departments; the Bureau of Public Roads can often help locate information for unusual types of cases by use of its severance effects bank maintained on EDP or by use of the Index. Sometimes the findings from land economic studies need to be aggregated and analyzed collectively. The conclusions listed in the following statements have been based on an aggregated analysis of cases in the Public Roads severance effects bank and narrative reports.

- Land values have increased from the estimated value for about four cases out of five.
- Special characteristics have been associated with the remainders for which sales prices were more than the average estimated value. Some of these characteristics of the remainder parcel are: (1) nearness of the remainder to an interchange area; (2) remainders caused by Interstate highway takings; and (3) use of the remainder for commercial purposes.
- Owners of landlocked remainders are not damaged to the extent anticipated at the time of settlement.
- Owners of vacant property are more likely to gain more from the sale of remainders than owners of residential or agricultural properties.

Narrative Reports

In addition to the nearly 4,000 sales (more than 3,000 cases) recorded in the Public Roads severance bank, approximately 2,900 narrative case studies for 2,262 remainder sales have been processed. These narrative reports have been listed and compared in *Narrative Reports of Highway Severance Effects: Index and Summary.* This Index should make it easier for an appraiser to obtain information about a comparable situation from printed studies. The appraiser can check the Index to determine whether a narrative report is available for a situation similar to his current assignment. If so, he may obtain a copy of the report from his State highway department library, the State highway department originating the study, or from the Bureau of Public Roads.

The appraiser can refer to the Index to determine whether a severance study has been made on a landlocked parcel near an interchange for which the value has been within a specified range. Or, an appraiser having an assignment concerning leaseholds would learn from the Index that severance studies involving leaseholds have been reported by Florida, Colorado, Georgia, and Illinois. Narrative reports involving severance studies for church and school properties have been made by Michigan, Texas, Nebraska, California, Florida, Kentucky, Illinois, Ohio, and others. Some other special severance studies listed in the Index include land used for a boys home, a hunting camp, a drive-in theater, a trailer park, a golf course, a zoo, a gun target range, and a dam project, a remainder selling for corporate stock, a sale of more than a million dollars, and a remainder that had a loss of a lake view.

This Index will not end the need for the Public Roads severance bank of cases as approximately half the cases in the bank have not been reported in narrative form. But the Index should make information more readily available on the 2,900 severance cases covered in narrative reports. Such information should help bridge the gap between severance effects studies and the application of the study findings.

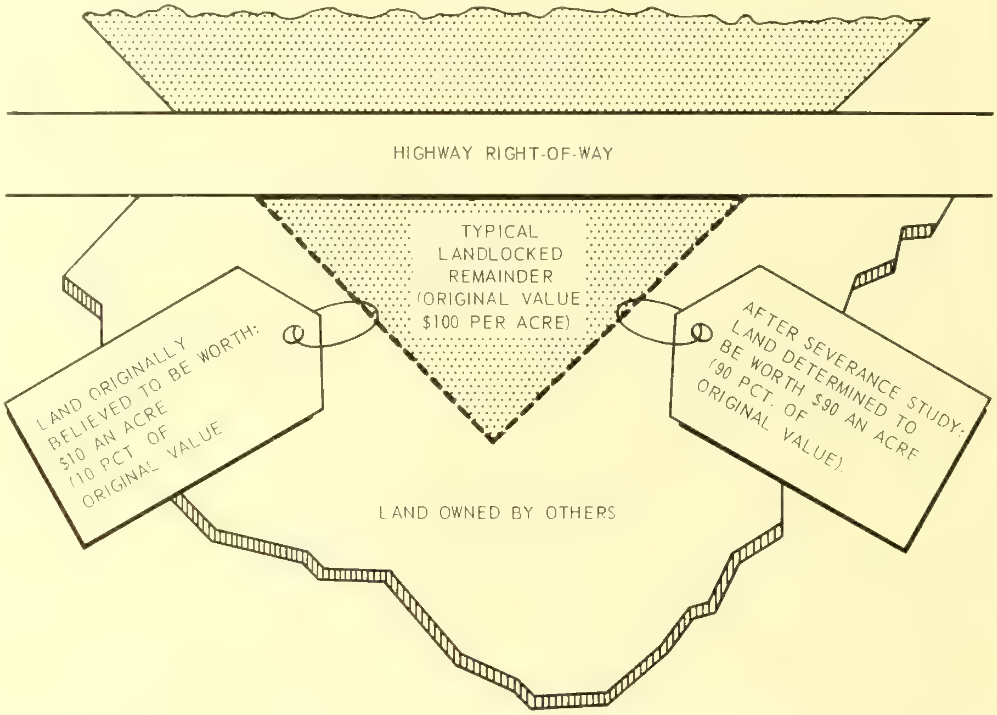
HIGHWAY RIGHT-OF-WAY
TYPICAL LANDLOCKED REMAINDER (ORIGINAL VALUE $100 PER ACRE)
LAND ORIGINALLY BELIEVED TO BE WORTH: $10 AN ACRE (10 PCT. OF ORIGINAL VALUE
AFTER SEVERANCE STUDY: LAND DETERMINED TO BE WORTH $90 AN ACRE (90 PCT. OF ORIGINAL VALUE).
LAND OWNED BY OTHERS

Figure 1.—Savings possible from use of findings from studies on severance effects.

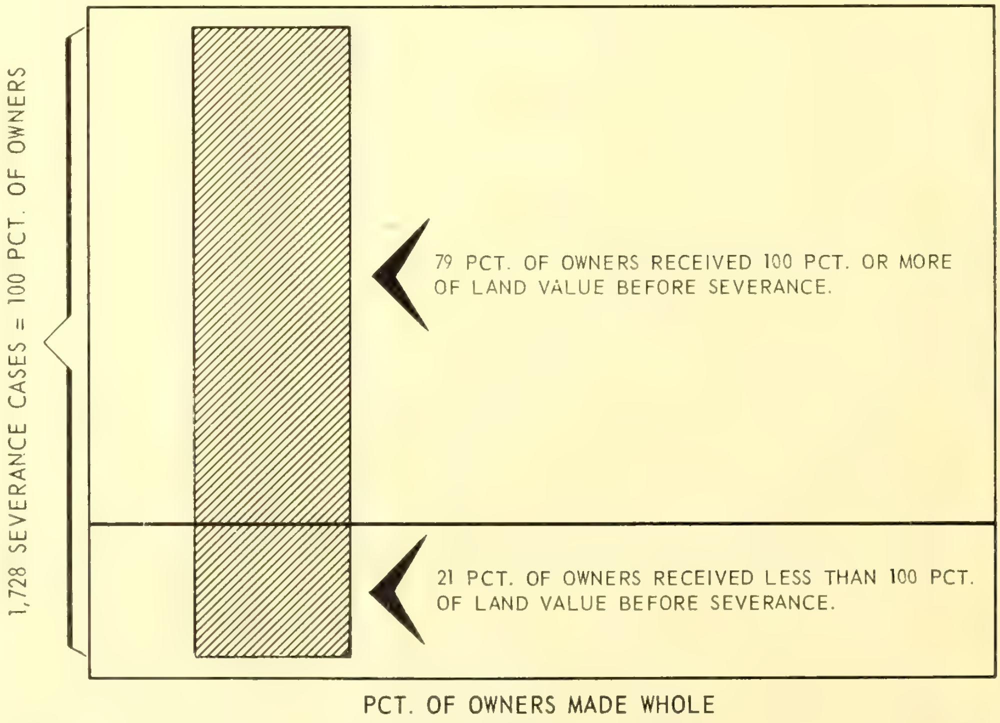

Figure 2.—Percentage of owners made whole and percentage who lost based on 1,728 severance effects cases.

Overall Analysis Needed

Shortening the gap between study findings and application to similar situations might also be accomplished by analyzing several studies and making the results generally available. The need for such an analysis was stated by Rudolf Hess, of the California Division of Highways, before the American Institute of Real Estate Appraisers, meeting in Los Angeles in November 1964, as "The remainder parcel sales data collection is one of the first direct research links between the appraisal profession and land economics. The analysis of remainder parcel sales data may provide the vehicle for a firmer marriage of these two specialized areas; it may provide the means for appraisers to more conveniently and confidently consider land economic data and theory. Collective analysis is the key. Collective analysis is a tool of land economics. . . ."

Several States have used overall analyses of severance studies. For example, A. B. Grace of the Texas Highway Department in an article, *Severance Damage Studies*, Right-of-Way, vol. 13, No. 1, February 1966, pp. 55–60, presented general findings on the Texas experience with severance studies. Mr. Grace believes that severance effects studies should not be used to ". . . advocate the development of appraisal formulas . . ." but ". . . as a source in locating comparable sales and as general information in predicting the change in use of a particular remainder."

Some analytical work has been done by Public Roads on both the 4,000 sales recorded in its severance bank of cases and the 2,262 sales for which the States have issued narrative reports. A few of the more important findings from these analyses are described here.

Findings from the Public Roads bank of severance studies and the narrative reports are very similar, as might be expected when the same case is reported in the bank and the narrative report. Approximately 60 percent of the narrative reports are for severance cases in the bank and nearly 40 percent of the cases in the bank are also reported in narrative form. The agreement of the findings from the two sources is shown by approximately the same percentage of owners (about 80 percent) receiving more than 100 percent of the original value of their land. Better than average agreement is also present in the findings concerning remainders near interchanges, remainders used for commercial purposes, and takings for Interstate highway routes.

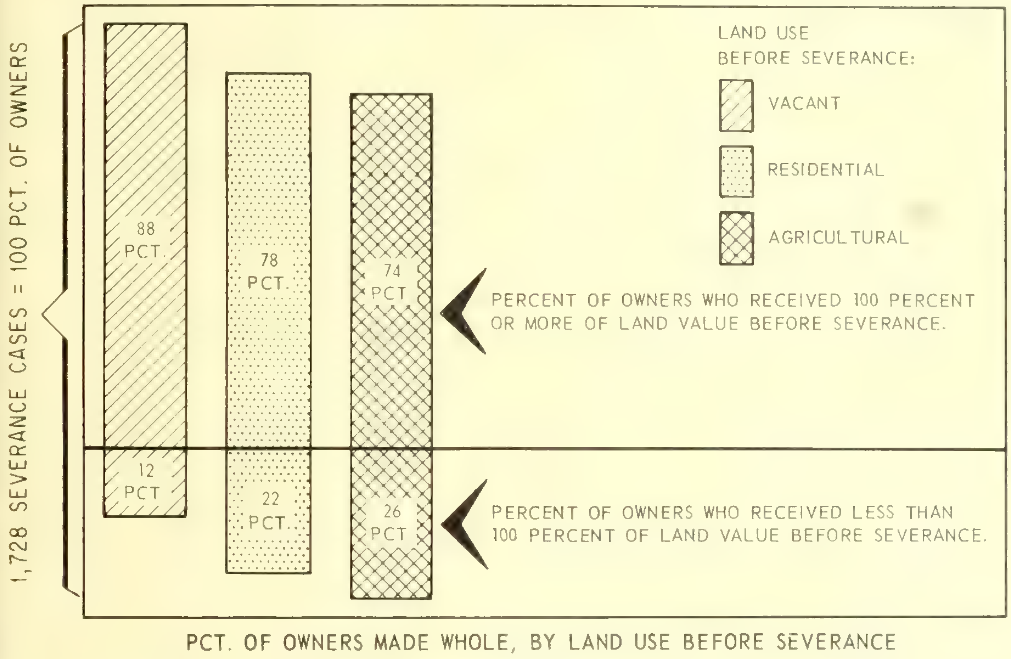

Figure 3.—Land use before severance, by percentage of owners made whole, based on 1,728 severance effects cases.

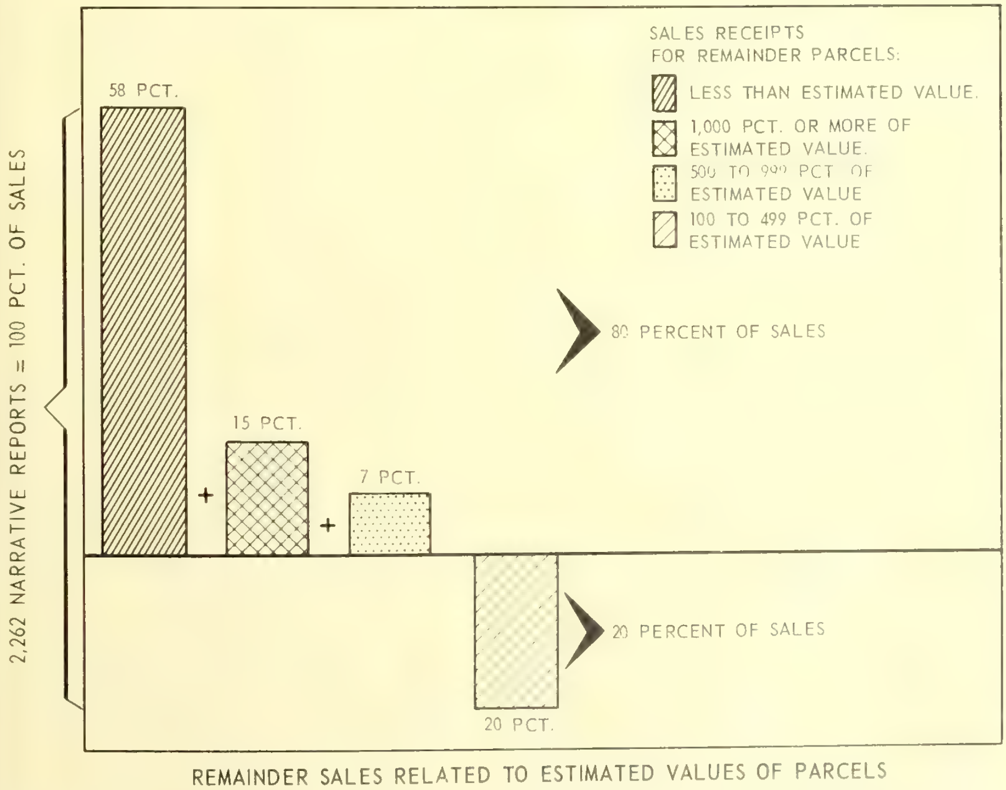

Figure 4.—Remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

Severance Bank Findings

Many remainder parcel sales involve only a part of the remainder after a taking for highway right-of-way. When any part of the remainder has been sold, a preliminary comparison of the per acre value of the parcel sold can be made with the per acre value of the entire property at the time of severance. Such a comparison obviously cannot be conclusive; but its use makes possible a per acre comparison of the value of the part of the remainder that has been sold to the value of the entire property. The parcels that have been sold may have been more or less desirable than the entire property or the entire remainder. Such a comparison shows what happened to land values rather than to owners and does not show whether owners were benefited or damaged. However, such a comparison permits some tentative findings about highway severance effects in many situations where only part of the entire remainder has been sold. Thus a recovery rate can be determined by dividing the value per acre or per square foot of part or all of the remainder that has been sold by the per acre value at the time of the highway taking. Thus a recovery rate of more than 100 percent means that the remainder has had an increase in unit value. Conversely, when the rate is less than 100 percent, the remainder has declined in value. A recovery rate of more than 100 percent for remainder sales has been shown in most severance studies. Less than one-fourth of the remainder sales in the Public Roads bank had a recovery rate of less than 100 percent. About one-fifth had recovery rates of 500 percent or more and one-tenth had recovery rates of 1,000 percent or more.

Because of the extremely large increases in per acre values for some remainder parcels, simple arithmetic averages may not be a satisfactory summary measure of the typical recovery rate for severed land parcels. Median values provide another way of summarizing the overall recovery rate. As a median is a middle value with half of the cases above and half of them below, those remainder parcels having extremely large recovery rates do not have such a noticeable effect on median values as they do on average values. Thus, for the nearly 4,000 sales in the Public Roads severance bank of cases, the median recovery rate is 160 percent and the average value is 495 percent.

When sales of remainders are made after a lapse of time (1 year or more) after the taking, allowance should be made for general land value changes in the area. For this adjustment, an average increase in value of 7 percent per year was used for data reported here. Where this adjustment was applied, the overall median recovery rate was approximately 140 percent. For sales that are made 2, 3, or more years after the taking, recovery rates are larger than the overall recovery rate, even after the adjustment has been made for general land value increases. For example, the adjusted recovery rate for remainder parcels sold more than 3 years after the taking was nearly 170 percent. Apparently the value of remainders tends to go up more than land values generally, at least for 3 or more years following the right-of-way taking.

Comparison of the recovery rates for different types of remainders—for example, landlocked remainders shows some significant variations. Damages to landlocked remainders previously have been estimated at 90 percent or sometimes at 100 percent. An estimate that the value of a remainder parcel will be damaged 90 percent means that the estimated recovery rate is only 10 percent. On the basis of cases in the Public Roads severance bank, such an estimate is seldom realistic. Data on the 94 landlocked cases in the bank suggest that landlocked parcels are damaged about 8 percent and that the median recovery rate for landlocked parcels is 92 percent. Landlocked remainders, however, are damaged more than other types of remainders. All types of remainders in the severance effects bank had a median recovery rate of 160 percent and an average recovery rate of 495 percent. Also more landlocked remainders are damaged—have a smaller recovery rate—than other remainders. About half of the landlocked remainders had recovery rates of less than 100 percent but only about one-fourth of the other remainders had recovery rates of less than 100 percent.

Special analysis has been made of changes in value of remainder parcels near interchanges, especially interchanges on the Interstate system. About 30 percent of the remainder cases in the Public Roads bank are located within a half mile of an interchange, a distance often used to distinguish between areas called interchange areas and noninterchange areas. As might be expected, recovery rates for remainders located within a half mile of the interchange area are better than the recovery rates for comparable remainders located farther away.

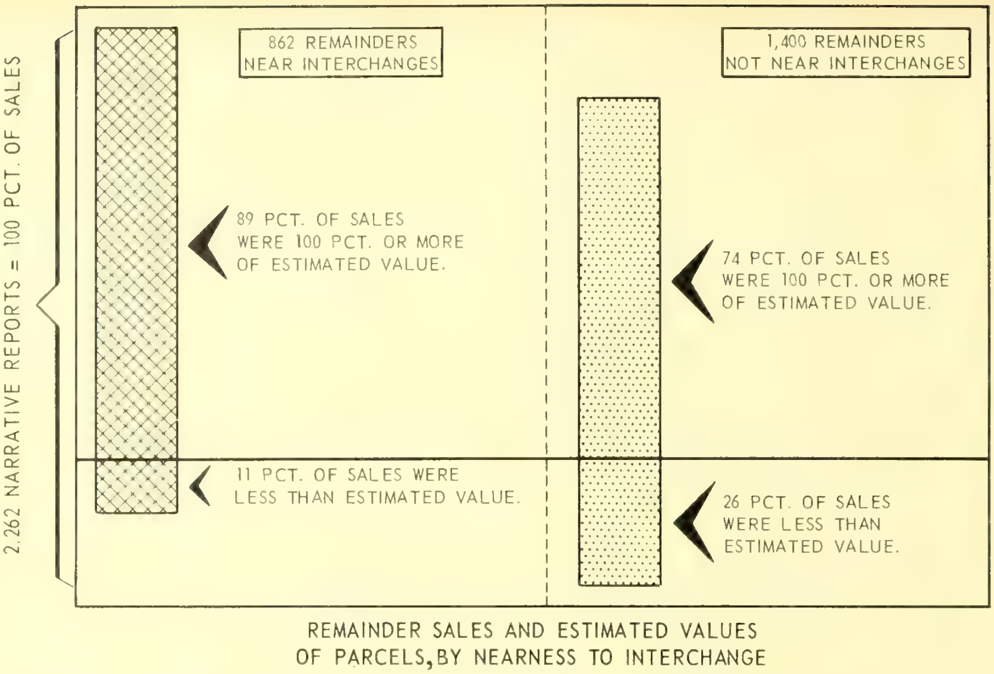

Figure 5.—Remainder sales and estimated values by nearness of remainder to an interchange area, based on 2,262 narrative reports on severance studies.

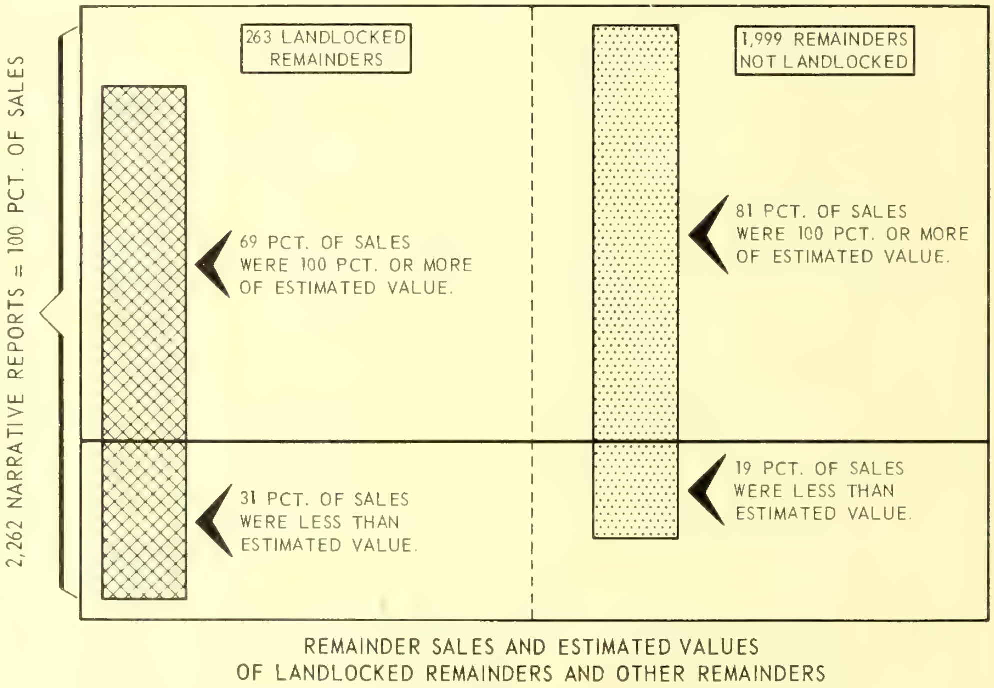

Figure 6.—Effect of remainder being landlocked by remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

Owners Made Whole

The entire remainder parcel has been sold for more than 50 percent of the cases in the Public Roads severance bank. From these cases, enough facts are available for a determination as to whether the owner is in as good a position as he was before the taking for right-of-way acquisition. To make this determination information is needed on (1) value of the property before right-of-way acquisition, (2) payment to the owner for the part taken for the highway and estimated damages to the remainder, and (3) receipts for the entire remainder. When the total receipts, for the right-of-way taken plus the amount received for sale of the remainder, amount to as much as the estimated property value before the taking, the owner can be said to have been made whole. For the 1,728 cases in the Public Roads bank for which the entire remainder has been sold, approximately 4 out of 5 property owners were made whole. The other property owners had less money after sale of the entire

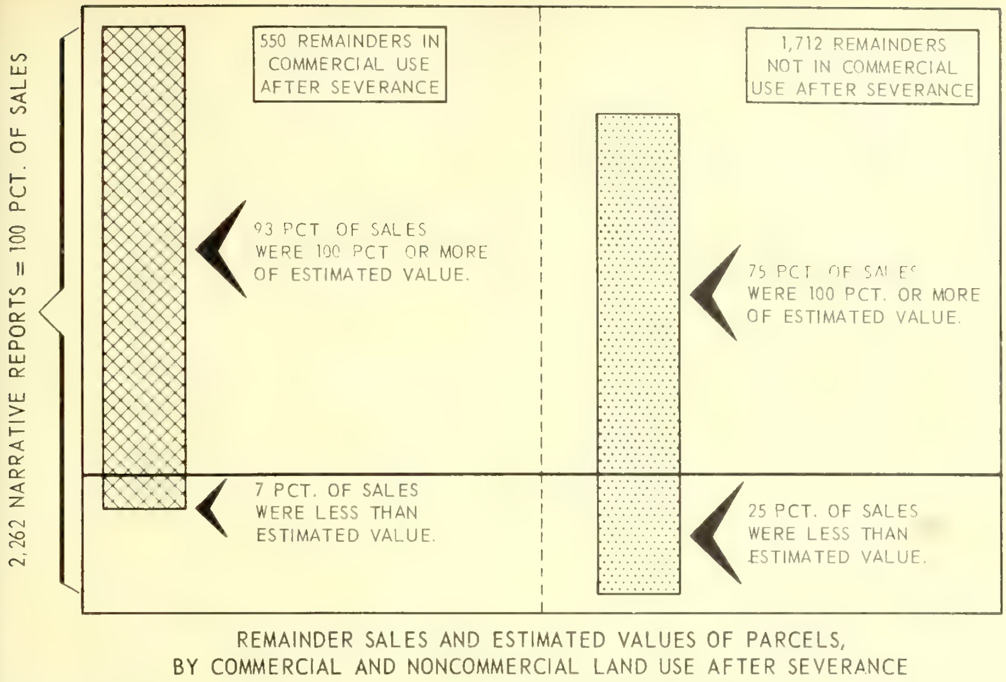

Figure 7.—Effect of commercial use by remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

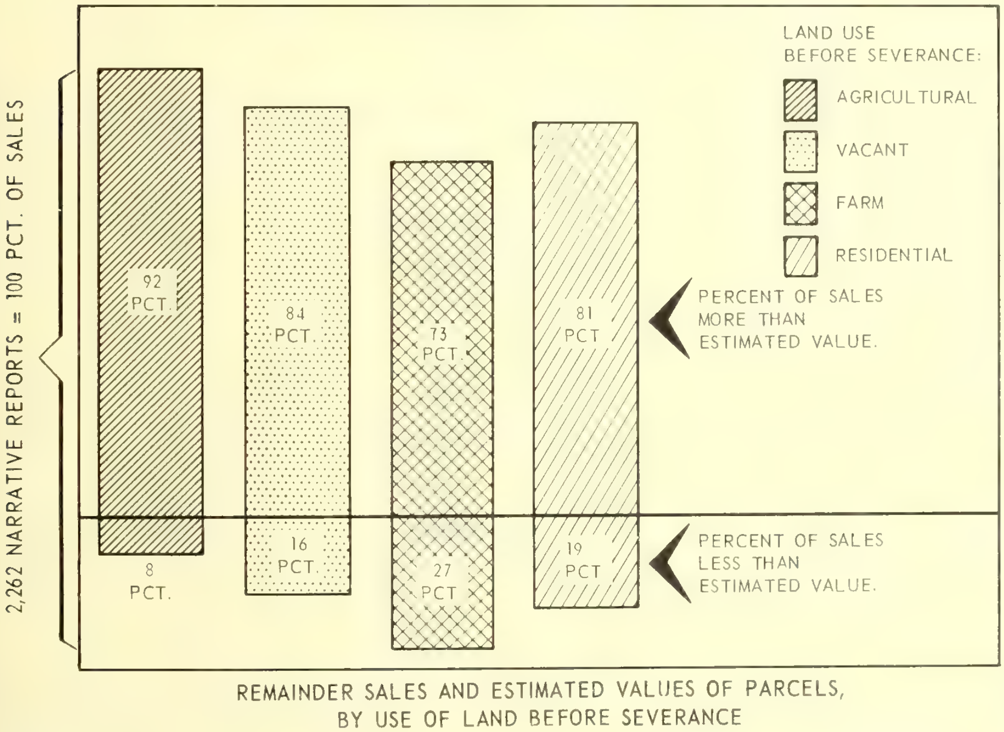

Figure 8.—Effect of land use before severance on remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

remainders than the value of their property at the time of severance, as shown in figure 2. Some owners received total receipts amounting to several times the worth of their property before the taking for the highway.

An analysis of the bank case data on owners not made whole—those who had receipts less than the original value of their property—shows that most of the losses were moderate. For nearly half of the owners who lost money, the loss was not more than 10 percent of the original value of the property. In 5 percent of the cases where the landowners had losses (1 percent of all sales recorded in the bank) the amount received was less than 50 percent of the original value. Efforts to close the gap between findings from severance studies and application of the findings should help prevent inadequate payments for right-of-way; this is a problem of as much concern to conscientious highway builders as excessive payments.

Effect of land use

Use of the property before severance, whether vacant, residential, or agricultural, affected the receipts of the owners for cases in the Public Roads bank. Owners of vacant land had receipts amounting to 135 percent of the before value of their property. Owners of residential and agricultural properties had receipts equaling 110 percent and 116 percent of the before value of their property. Owners of vacant property generally fared better than owners of residential or agricultural property. A partial explanation of the value increases of vacant land is that it is more frequently put to a more profitable after use than is residential and agricultural property.

As more experience has been gained in relation to right-of-way acquisition, the potential value of vacant property has been recognized. Data now available from the bank show that a smaller percentage of owners of vacant property—49 percent—are being paid damages than owners of agricultural and residential property—67 percent each. Latest available information on percentages of owners paid damages and the percentage having apparent losses, by land use before the highway taking are, respectively: (1) vacant, 49 percent and 12 percent; (2) agricultural, 67 percent and 26 percent; and (3) residential, 67 percent and 22 percent. Data combined for these three uses of land before the highway taking show that 61 percent of the owners were paid damages and 20 percent apparently had losses. The relation of receipts to estimated damages for these land uses before the taking is shown in figure 3.

Narrative Reports

When a group of narrative reports has been assembled, analyses of the findings can be made and general comparisons noted as to which types of remainder parcels have been sold for a gain or a loss. Only a general indication of gains or losses can be obtained from aggregating narrative reports. State laws or practices differ as to the bases used for determining damage payments, the different uses of control areas, and the different methods for adjusting estimates in relation to general changes in land use. The information taken from the narrative reports permits a comparison of estimated values of remainder parcels and sales prices. Thus an indication can be obtained of the experience that owners have in relation to the sales of remainders. Generally, owners of remainder parcels that were sold for more than the estimated value can be said to have benefited. By the same reasoning, when the remainder was sold for less than its estimated value, the loss can be equated generally with a loss by the landowner.

Four-fifths of the sales reported in the 2,262 narrative cases exceeded the estimated values of the parcels sold. Also, for more than half of the narrative cases reported, sales prices exceeded the estimated values of the remainders by 50 percent or more. Large gains of 500 percent or more of the estimated value were noted for 15 percent of all narrative sales. Conversely, 20 percent of the remainder sales reported in the narrative cases were for less than the estimated value of the remainder. The relation between estimated values and remainder sales is shown in figure 4.

Nearly 900 cases for which there are narrative reports were for remainder sales for land near interchange areas. Nearly 90 percent of the remainder sales for parcels located near an interchange exceeded the estimated value of the remainder. But only about 75 percent of the remainder sales for parcels located away from an interchange area exceeded the estimated value. Also, larger gains in sales of 500 or 1,000 percent of the estimated value were more pronounced for interchange areas. This is shown in figure 5. These differences in relation to nearness to an interchange are significant at the 95 percent level of confidence.

Because of problems related to appraising the value of landlocked remainders when right-of-way for a controlled-access highway is acquired, special attention has been given to reports for this type of remainder. More than a tenth of the sales—262 of the 2,262 cases reported in narrative form—are landlocked remainder parcels. Sales for nearly a third of these remainders were less than the estimated values. At the same time, sales prices greatly exceeding estimated values are received more often for landlocked remainders than for other remainders. These findings—gains and losses for landlocked remainders in excess of those for other remainders—emphasize the problems associated with appraising landlocked remainders. The relation of sales and estimated values for remainders landlocked and those not landlocked is shown in figure 6.

Experience and logic indicate that land use and land value are positively related. Thus, if the ultimate use of a remainder can be determined at the time of the highway taking, this information can be useful in estimating the value of the remainder. For example, remainder parcels that are used for commercial purposes after the severance seem to increase in value more than remainders used for other purposes. This is substantiated by the fact that of 550 remainders in commercial use after the taking, 12 percent sold for 1,000 percent or more of the estimated value. Also, only 7 percent of the remainders used for commercial purposes sold for less than the estimated value, but 25 percent of the properties not used for commercial purposes sold for less than the estimated value. These relations are shown in figure 7. The use of the property at the time of the highway right of-way acquisition also is an indicator of the possible value of the remainder. Vacant land or land in commercial use is more likely to increase in value than land in residential or farm use. More data on such situations are shown in figure 8.

Application of Statistical Concepts to Accident Data

(continued from p. 137)

Office of Technical Services PB 151055, June 1958.

(2) *Application of Statistical Quality-Control Techniques to Analysis of Highway-Accident Data,* by Monroe L. Norden, Jesse Orlansky, and Herbert H. Jacobs, HRB Bulletin 117, Statistical Analysis of Highway Accidents, 1956, pp. 17–31.

(3) *Operational Route Analysis,* by Burton M. Rudy, HRB Bulletin 341, Accident Analysis and Speed Characteristics, 1962, pp. 1–17.

(4) *Two Simple Techniques for Determining the Significance of Accident-Reducing Measures,* by Richard M. Michaels, PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, vol. 30, No. 10, October 1959, pp. 238–239.

(5) *Daylight "Lights-On" Plan by Port of New York Authority,* by Edmund Cantilli, Traffic Engineering, vol. 36, No. 3, December 1965, p. 17.

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Economic Study of Luminaire Mounting Heights

(continued from p. 144)

Actual installation of floodlighting systems would be necessary before the system could be fully evaluated as to effectiveness and economy. But, interchange floodlighting costs, shown in table 7, seem to be about equal to the costs of a conventional, 30-foot mounting height design. Also safety and esthetic considerations favor floodlighting because fewer poles are needed in this design. However, studies to determine the pavement brightness, glare, and effectiveness in fog or on wet pavement are needed to evaluate floodlighting or tower-lighting. Governing conditions, such as the type of property development adjacent to the highway, the highway geometrics, and the personal choice of the decision-maker, influence the cost-effectiveness evaluation of a specific lighting installation. Additional information regarding the differences in design criteria and field measurements would affect the final decision.

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Highways and Human Values, the Bureau of Public Roads, U.S. Department of Commerce, annual report for fiscal year 1966, contains a review of the accomplishments of Public Roads with particular attention given to the social responsibilities inherent in the State and Federal-aid highway programs. *Highways and Human Values* is available from the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402, for 75 cents a copy, prepaid.

The illustrated publication points out the concern of the Bureau of Public Roads and the State highway departments for the total impact of highways on all the elements of life. A broad picture is presented of the many social values, such as highway safety and beautification, involved in the National System of Interstate and Defense Highways and other Federal-aid highway programs.

The 1966 annual report also reviews the progress of the Bureau of Public Roads in conservation and recreation and describes the highway construction in national forests, parks, and on other Federal lands that is done under the direct supervision of Public Roads. Foreign activities including construction, financing, and technical assistance are also outlined.

The revitalized interest in the human values associated with highway progress is illustrated by the change in size and presentation of the 1966 annual report; *Highways and Human Values* has been designed in both copy and format to be of more interest to the general public. Many photographs illustrate the extensive work being done by the Bureau of Public Roads to satisfy the needs of transportation as well as society. Statistical tables that cover the progress and status of the Federal-aid and allied highway programs for fiscal year 1966 are published in a supplement. The *Supplement to Highways and Human Values,* also can be purchased from the Superintendent of Documents; it is priced at 25 cents, prepaid.

Presplitting, A Controlled Blasting Technique for Rock Cuts

Presplitting, A Controlled Blasting Technique for Rock Cuts is a 36-page research and development report presenting a state-of-the-art summary of a relatively new controlled blasting technique that will be of interest to State highway engineers, contractors, manufacturers, consultants, university engineering personnel, and others engaged in rock excavation activities. The report is available from the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402, for 30 cents a copy, prepaid.

The findings of a staff study by the Bureau of Public Roads on rock presplitting are presented, particularly as they apply to highway construction. The different field practices used to improve rock blasting operations are reviewed, and the use of presplitting is emphasized as a method of producing smooth wall surfaces that require less maintenance and provide more safety during highway construction.

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Thomas Creek Bridge on the Oregon Coast Highway, U.S. 101, Curry County in Southern Oregon.

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Aerial Analytic Triangulation Investigation, Wyoming Interstate 80

BY THE OFFICE OF
ENGINEERING AND OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by [1,2] JESSE R. CHAVES,
Highway Engineer, Aerial Surveys Branch,
Highway Standards and Design Division

The potential for the use of analytic (mathematical) aerial triangulation to establish supplemental control points for large scale highway mapping can be developed, according to the results of the investigation reported in this article. At present, investigation of all facets of photogrammetry as aids for highway location, route selection, design, and construction is being urged. As more use is made of aerial photography, a more expeditous method of mapping from aerial photographs is being sought. The analytic aerial triangulation method, in which computers are used to establish supplemental control points and thereby curtail the number of ground survey points otherwise required, has been applied to small scale mapping of projects to a limited degree. But, little investigation has been made as to the accuracy that might be obtained if this method were to be applied to large scale mapping of projects.

Although aerial analytic triangulation is a potentially useful method, operational techniques and hardware should be refined, according to the author. He notes that present aerial cameras are neither designed nor calibrated for analytical photogrammetry. To take full advantage of the mathematical techniques, some slight design changes in aerial cameras and related equipment will be needed.

Introduction

INTEREST OF highway location and design engineers in the use of analytic (mathematical) aerial triangulation prompted the investigation reported here on this supplemental control used in photogrammetric mapping. Conventional analog aerial triangulation with first order bridging instruments and ground control surveys heretofore have been used to provide the ground position and elevation points—the supplemental control—needed for the absolute orientation of stereoscopic models in photogrammetric instruments for large scale topographic mapping with small contour intervals. Now electronic digital computers have made it possible to use bridging for supplemental control for small scale topographic mapping. Although analytic bridging was understood and used by a few engineers before 1960, only recently have highway engineers become interested in its application to highway engineering mapping. A need existed, therefore, to evaluate the analytic approach for extending surveyed ground control to determine how accurate the computed control data would be for aerial mapping by photogrammetric equipment and materials presently available to highway organizations.

The decision to investigate the use of analytic aerial triangulation for highway surveys was made because:

- Conventional ground control surveys are expensive, time consuming, and sometimes difficult to make. Analytic extension of ground control points would minimize the need for so many ground control points and limit the number of positions and elevations that would have to be surveyed on the ground.
- Electronic digital computers are available for use of most highway organizations.
- The monocomparator required in the aerial analytic triangulation procedure to measure the x and y photographic plate coordinates can be bought for much less than the conventional analog, optical train bridging instruments. Also less training is required for the monocomparator operators to successfully measure the x and y coordinates of image points on glass plate transparencies of the aerial photographs than for the operation of the optical train bridging instruments.
- The aerial analytic triangulation method has the potential required to provide data for accurate photogrammetric mapping for highway location and design. Also, this method is potentially more flexible than the conventional analog types of bridging instruments.

Preliminary Evaluation

Results obtained in a preliminary evaluation of analytical photogrammetry encouraged the author to continue the investigation reported here. The mathematical procedures developed by the Coast and Geodetic Survey (*1*)[3] were used in the preliminary evaluation; they were modified slightly and programed in the Fortran language. The mathematical procedures (*1*) have since been replaced by a method called three-photo aerotriangulation (*2*). The preliminary evaluation (*3*) of the analytic method was made in 1964 on an 18,000-foot segment of Interstate Highway 66 in Fairfax County, Va., and reported by the author in his master's thesis, *Survey Control Extension by Analytic Aerotriangulation for Highways*, submitted to Syracuse University, September 1965. In this work, seven photographs, taken with a Wild 6-inch focal length aerial camera at a scale of 1:8,400, were analytically bridged. A monocomparator was used to measure x and y coordinates of natural images and targeted points on the photographic glass plate transparencies of the aerial photographs. Second-degree cantilever strip adjustments were made by using three horizontal and six vertical control points. The resultant root mean square errors (RMSE) on test points were 0.41 foot for the horizontal and 0.71 foot for the vertical ground coordinates.

Objectives

The results of the preliminary evaluation of the analytic method encouraged the author to undertake the research reported in this article. Eight major objectives were established for the investigation of aerial analytic triangulation on Wyoming Interstate Highway 80, as described in the following statements.

- To analytically bridge 11 photographs (10 stereoscopic models) taken at a scale of 1:6,000—500 feet to 1 inch—with a 6-inch focal length aerial camera lens.
- To evaluate the effect of analytically bridging photographs that had been drilled with a Wild PUG point transfer device.
- To develop computer programs written in the Fortran language, to perform a coordinate transformation of image points on the photographic glass plates that have side fidu-

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[1] Presented at the 46th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1967.

[2] Mr. Chaves is now a Highway Engineer in the Engineering Systems Division, Office of Research and Development.

[3] Italic numbers in parentheses identify the references listed on page 159.

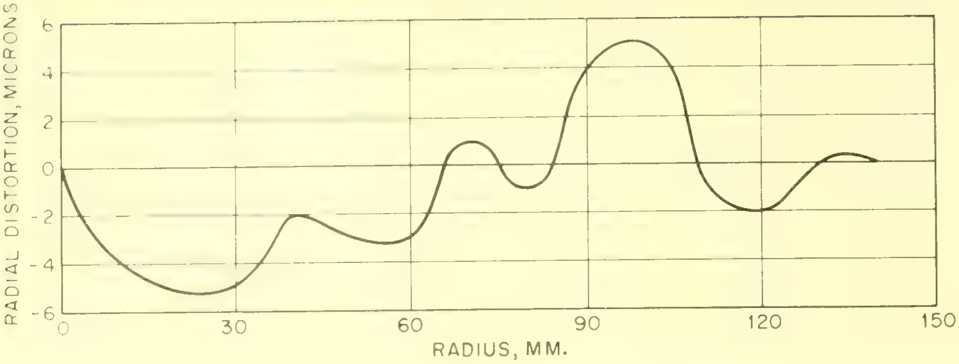

Figure 1.—Average radial distortion curve for Zeiss Pleogen lens.

cial marks, and to apply a polynomial curve-fitting technique to compensate for the effects of radial lens distortion.

- To determine the density and distribution of ground control needed for adequately adjusting a strip of 10 stereoscopic models.
- To determine the degree of strip adjustment needed for a strip of 10 stereoscopic models.
- To analyze photographic materials and photogrammetric instruments, equipment, and methods as possible sources of error in the analytic system of bridging.
- To revise the cantilever adjustment program originally written for use on an IBM 1401 for the IBM 7010 system.
- To make recommendations for improving and implementing the analytic method and suggest needed research.

Conclusions and Recommendations

The aerial analytic triangulation method is potentially useful as a method for obtaining accurate supplemental data that can be used in the compilation of topographic maps for highway location and design, according to results obtained in the investigation reported here. The author cautions, however, that operational techniques and hardware should be refined to overcome the four main sources of error in the computation of the *X*, *Y*, and *Z* coordinates of selected points on the ground. These four sources of error that affected the accuracy of results obtained in the investigation were: Film deformation, pass point transfer, *x* and *y* coordinate measurements of points on the glass plate transparencies, and ground control.

Most present aerial cameras are neither designed nor calibrated for analytical photogrammetry. To take full advantage of mathematical techniques to compensate for errors, some slight design changes in aerial cameras and related equipment undoubtedly will be needed.

In future investigations of aerial analytic triangulation for highway mapping, the author recommends that consideration be given to use of the equipment and methods listed here: (1) color diapositive plates; (2) aerial cameras equipped with eight fiducial marks or a reseau grid; (3) stereocomparator (*4*); (4) photograph scales of 1:12,000 or smaller; (5) other mathematical methods of aerial analytic triangulation described in references (*2*, *5*, *6*, *7*, and *8*), and different point transfer and marking devices.

The conclusions and recommendations were based on the findings discussed in the summary at the end of this article.

Test Procedures and Results

Eleven photographs at a scale of 1:6,000 were selected from a flight strip taken in July 1963 by the Continental Engineers, Inc., Denver, Colo., for mapping a corridor for Wyoming Interstate Highway 80 between Green River and Rock Springs in southwest Wyoming. These 11 photographs covered a strip of topography that was approximately 4,500 feet wide and 20,000 feet long and that had a light to moderate brush cover. The photographs were taken, from an average flight height of 3,000 feet, with a Zeiss RMK A 15/23 aerial camera equipped with a Pleogon lens having a calibrated focal length of 152.45 mm. ±0.02 mm. and a maximum aperture of f/5.6. The average radial lens distortion based on determinations made on two radii did not exceed ±5 microns, as shown in figure 1. The distortions were determined within an accuracy of 2 microns. The distance between the fiducial marks in both directions was 226.00 mm. ±0.02 mm. The negative film used had an estar base from which diapositive plates (Kodak Aerographic Positive Plates, Improved, 0.25 inch thick) and photographic prints were made with a LogEtronics CP 18 automatic dodging printer.

Image selection

The image points used in the triangulation experiment were those for which ground control data were available. These control points had been surveyed for use in compiling topographic maps of a corridor along Interstate Highway 80 in Wyoming. The points measured were images of targets and images of natural objects that had been selected in accordance with the mapping needs of the project. Pass points for each stereoscopic model were selected in the usual rectangular pattern in the six classical locations. Two or three additional points were selected in the area of triple overlap of the photographs for each two adjacent stereoscopic models so as to ensure that enough acceptable points were available for the scaling of one stereoscopic model to another in the cantilever assembly. All image points, targeted and natural, used in the triangulation were predrilled with a Wild PUG 3-point transfer device equipped with drills having a diameter of 60 microns.

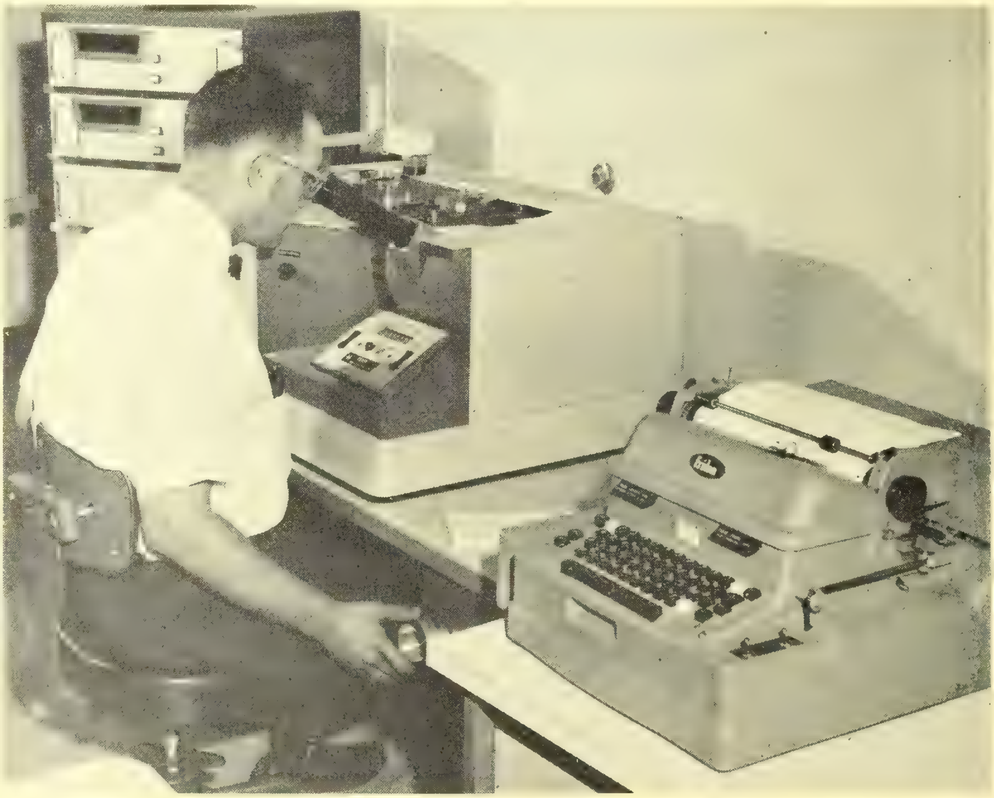

Figure 2.—Nistri monocomparator, model TA1/P and accessory equipment.

Table 1.—Errors in horizontal coordinates from third degree adjustments

Point	Errors in horizontal coordinates from third degree adjustments									
	1		2		3		4		5	
	X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
Number	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
45-1	0.95	−0.68	0.86	−0.74	0.91	−0.58	−0.96	−0.64	[1] −0.02	−0.05
SW-45	[1] −0.02	−0.09	[1] −0.05	−0.18	[1] −0.04	−0.09	[1] 0.00	0.14	0.71	0.53
SW-44	0.14	0.31	−0.02	0.18	−0.05	0.18	0.00	−0.19	−0.29	0.74
SW-43	0.31	0.45	[1] 0.10	−0.29	[1] −0.03	0.22	0.02	−0.33	−0.14	0.67
42-1	−0.29	−0.61	−0.44	0.47	−0.42	0.45	−0.42	0.52	−0.46	−0.97
42-2	−0.30	1.21	−0.38	−0.01	0.04	0.79	−0.28	0.88	−0.38	−1.22
SW-41	[1] −0.08	0.28	[1] −0.13	0.08	[1] 0.08	0.15	[1] 0.00	−0.07	[1] −0.13	0.22
41-2	−0.80	0.10	0.75	−0.09	−0.36	−0.36	−0.49	−0.32	−0.88	−0.22
40-1	−0.73	0.12	−0.69	−0.12	−0.38	−0.47	−0.50	−0.39	−0.89	−0.08
38-1	−0.88	0.91	−0.80	0.80	−0.30	−0.41	−0.42	0.46	−1.21	−0.37
39-1	0.23	0.21	0.29	0.07	0.77	−0.22	0.64	−0.18	[1] −0.08	−0.27
SW-38	[1] −0.01	−0.49	[1] 0.09	−0.56	−0.49	−0.97	0.40	−0.93	−0.49	−0.91
SW-37	[1] 0.05	0.30	[1] 0.02	0.37	[1] 0.00	0.02	0.00	0.01	−0.72	0.11
BMC-131	0.68	−0.01	0.71	0.33	−0.85	0.08	0.78	0.06	[1] 0.13	−0.09

[1] Horizontal control points used to adjust strips.

Point transfer and marking

The accuracy with which pass points are transferred to a photographic strip materially affects the results. Obviously, precise x and y measurements made for a point but inaccurately transferred are of no value. At the present time considerable research is being conducted on different aspects of the measurement procedure, including evaluation of the types of point transfer and marking devices (*9*). At present no general agreement exists among those concerned as to the efficiency and reliability of the different types of instruments and devices. When using a monocular comparator it is a practical necessity to use some form of point transfer and marking system. Point transfer should be precise and stereoscopically correct. Measurement of plate coordinates is considerably facilitated and the possibilities of blunders from misidentification of corresponding images are minimized. Targeted points, however, can be located and measured readily without their locations having been premarked on the diapositive plate.

Primarily because of the inexperience in use of the PUG point marking and transfer instrument, some errors were introduced into the analytic system on the test project. Determining the magnitude of these errors, however, was considered impracticable. Most of the drilled holes were nonuniform in shape and size. Some targeted control points were drilled off center. Observation made with the Kelsh instrument of each stereoscopic model in the strip of photographs showed lack of stereoscopic correspondence of the drilled holes with the ground surface for a significant number of the points. Based on this stereoscopic analysis, digging or floating points were not used as control points for the strip adjustments. A comparison was made between the sign of the errors in the computed elevations and their positions were noted in the stereoscopic models as being either on, above, or below the ground surface.

There was a definite correlation between the sign of the errors in elevation and the observed elevation of the holes in the stereoscopic models. No actual elevation measurements were made because a significant number of the drilled holes could not be seen in the stereoscopic model. However, the transfer of points is only one source of error known in the investigation reported here.

Measurements

The x and y coordinate measurements were made with the Nistri Monocomparator, Model TA1/P, which provided both digital readout and typewritten outputs. This monocomparator is shown in figure 2. The monocomparator had been calibrated a few months before measurements were made. The smallest reading possible on this monocomparator is 1 micron. The diapositive plates were measured with the emulsion side down under a 10× magnification. The objective lens on this particular instrument was equipped with a 20-micron measuring mark. Measuring marks of other sizes are available from the manufacturer of the instrument. The coordinate output of this particular monocomparator was in a left-handed system. Provision was made in the coordinate transformation program to change the coordinate system into a right-handed system, so that all coordinates increased along the y axis away from the observer and to the right along the x axis. A simple wiring modification can be made at the factory to produce output directly in the right-handed system. Measurements were made in an air-conditioned room at 72° F. Periodic checks were made for possible instability. The instrument and accessory equipment had excellent stability throughout the measurement operations. It took about 1 hour to measure an average of 25 drilled holes per photographic plate and the 4 fiducial marks. Each of the drilled holes was measured three times and each of the fiducial marks was measured six times. The mean of these measurements was accepted as true x and y coordinates for each point, respectively. The measured points were always approached with the measuring mark from the same direction to avoid the possibility of screw backlash, although screw backlash had been determined to be only 2 or 3 microns in magnitude.

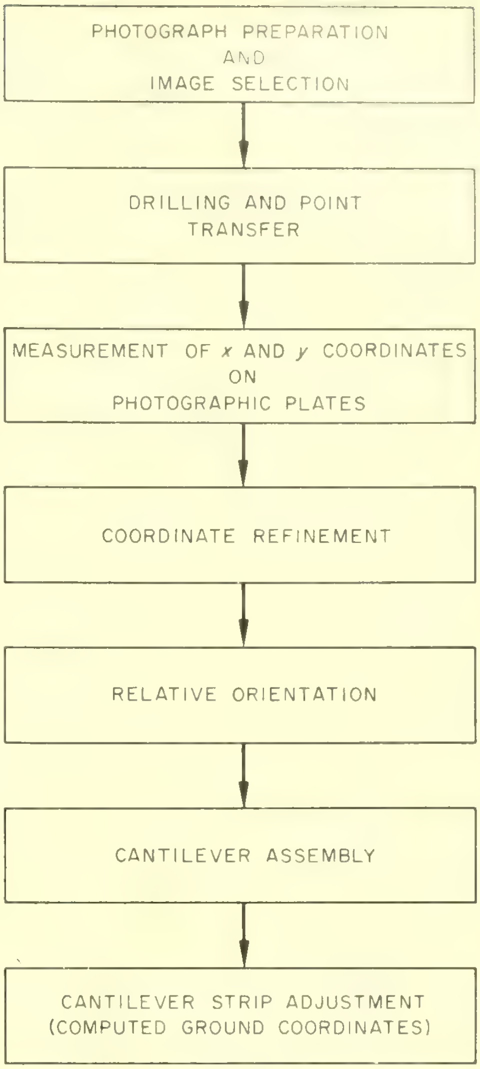
PHOTOGRAPH PREPARATION AND IMAGE SELECTION
DRILLING AND POINT TRANSFER
MEASUREMENT OF x AND y COORDINATES ON PHOTOGRAPHIC PLATES
COORDINATE REFINEMENT
RELATIVE ORIENTATION
CANTILEVER ASSEMBLY
CANTILEVER STRIP ADJUSTMENT (COMPUTED GROUND COORDINATES)

Figure 3.—Flow chart of aerial analytic triangulation.

Computations

Two electronic digital computers were used to make the mathematical computations of the analytic bridge. The cantilever strip adjustment program, which provided the X, Y, and Z ground coordinates of each measured point, was used in an IBM 7010 computer, which had a 60K digital storage capacity; all other programs were used in an IBM 1401, which had a 12K digital core memory.

Basic ground control was surveyed in a closed traverse approaching second order accuracy. Points identified in table 1 by the prefix SW were included in this traverse. All other ground surveyed points were assumed to be of at least third order accuracy, although no survey closure checks were actually made. The Electrotape and Tellurometer electronic distance measuring instruments and a Wild, T-2 Theodolite, were used to make the survey.

A generalized flow chart of analytic aerial triangulation used in the investigation reported here is shown in figure 3 and the basic computational concepts and procedures are described in the following paragraphs.

Certain mathematical operations must be completed to assure that the measured x and

y coordinates will be suitable for use in the analytic triangulation. These operations include the averaging of each set of coordinate measurements; converting the coordinate system from a left-handed to a right-handed system; performing a mathematical translation and rotation of the measured photographic plate coordinates; making compensation for film deformation; and correcting radial lens distortion.

Plate coordinate averages

The measurements made with the monocomparator are recorded in typed form. Card punching was performed directly from the typewritten record of the monocomparator measurements. Averages were computed with two separate computer programs for the three measurements made on each of the image points (drilled holes) and for the six measurements made on each of the fiducial marks. In the computer programs, the monocomparator measurements of the left-handed coordinate system were converted to a right-handed system by subtracting all x coordinates from an arbitrary constant.

When diapositive plates are placed on the monocomparator stage for measurement, the x and y axes of the photographic plate must be oriented physically parallel to the respective axes of the monocomparator, or some means must be provided for mathematical rotation so the two coordinate systems will be coincident. The relation of these axes is shown in figure 4. The orientation of a plate so that its axes are precisely parallel to the monocomparator axes is a time consuming procedure. Consequently, it is expedient to place the plate on the monocomparator so that the respective axes are more or less parallel, and then mathematically translate and rotate them to coincidence. The mathematical procedure is based on the instrument measured coordinates of the measured fiducial marks. The mathematical rotations are made by using standard rotation equations from analytic geometry. The resultant translated and rotated coordinates for all points measured on each photographic plate can then be referenced to the principal point of the photograph.

Film deformation

Plastic films are subject to dimensional change between the time of photographic exposure in the aerial camera and the printing of diapositives on optically flat glass plates. Therefore, some means of compensating for the movement of images is necessary. For cameras equipped only with four side fiducial marks, the only compensation possible is a comparison of the distances between the marks in two directions on the printed diapositive with those in the aerial camera. The distances between fiducial marks in the aerial camera may be furnished by the manufacturer or it can be measured on a diapositive plate (flash plate) that previously has been exposed directly to the aerial camera. Two scale factors were developed for each plate based on the distances between the fiducial marks reported by the manufacturer and those determined for each plate in the x and y directions. The x and y coordinates of all measured points were then multiplied by the respective film deformation correction factors.

Results obtained by comparing distances between fiducial marks in both directions on the measured photographic plates with distance between the same marks recorded on the camera calibration certificate were remarkably uniform in the deformation of estar base photographic film. The dimensional change in both the x and y photographic plate axes was about the same, as shown in table 2. All the computed linear factors for film deformation were less than unity, which indicated the estar base photographic film had expanded rather than shrunk in both directions. The average computed factors from all of the x and y measurements of fiducial marks on the separate photographic plates were 0.999676 and 0.999746, respectively.

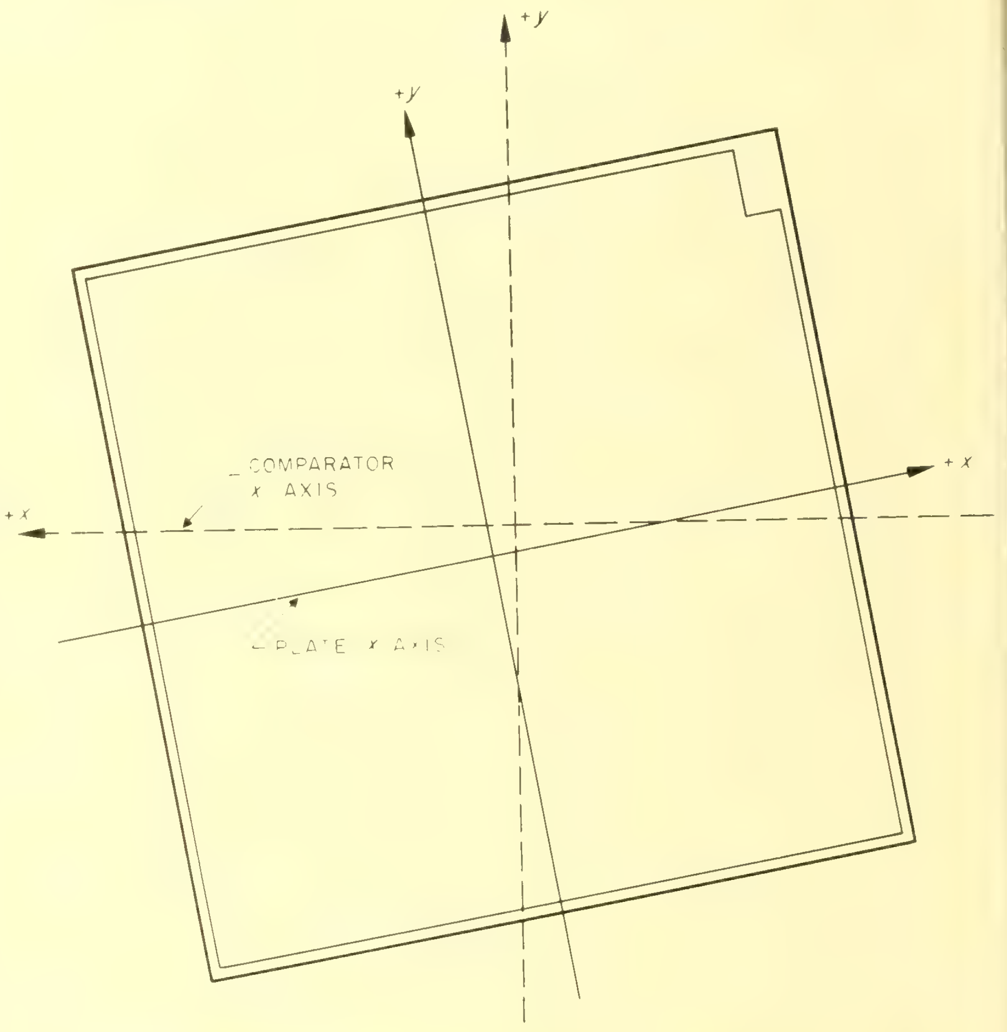

Figure 4.—Relation of diapositive plate to monocomparator axes at time of measurement.

This method of correction, based on the distance between side fiducial marks is not as accurate a method as would be desirable because film deformation is random and nonlinear. No better method could be devised for compensating for film deformation when only the four side fiducial marks could be measured. Film deformation is one of the sources of error in the analytic system of aerial triangulation. Recently developed, scale stable base films, such as estar base, are a factor in helping to curtail film deformation and thus minimize this correction method as a source of error.

One possible solution to the problem of film deformation would be the use of glass exposed directly in the aerial camera. This would eliminate need for film distortion compensation. However, no better accuracy in locating points can be expected as glass plate cameras use smaller formats and position accuracy is proportional to the scale of the photograph. The use of reseau equipped cameras might improve the accuracy (*10*). However, some practical considerations at present limit the use of these two techniques. One approach currently being employed to improve accuracy is the use of aerial cameras having eight rather than four fiducial marks. This permits more adequate mathematical restitution of points displaced by film movement (*11*). The distance between fiducial marks reported by the manufacturer of the camera used in this project was only to an accuracy of ±20 microns, and the diameter of the fiducial mark holes was 250 microns. Locating the precise center of such fiducial marks with a measuring mark only 20 microns in diameter was somewhat uncertain.

The average radial lens distortion curve for the Zeiss Pleogon lens is shown in figure 1. Positive lens distortion covers the displacement of an image radially outward from the center of the photograph; when distortion is negative the displacement is radially inward. Corrections for lens distortion must be made to the x and y coordinates of each measured point on the photograph.

An equation for the lens distortion curve shown in figure 1 was determined by a polynomial curve fitting program. The program used was one developed by W. R. Graves, IBM Corporation and modified by R. Larson, BPR, Region 15, and the writer for use on the IBM 1401. This program generates an approximating polynomial using the least squares technique. Coefficient terms of the polynomial curve are determined and they are used to compute the amount of radial lens distortion for any given radial distance from the center of the photograph. Actual computation of the distortion is accomplished by a short program in which the appropriate coefficients are used. The x and y coordinates of each point are then corrected for the effects of image displacement caused by lens distortion. The particular distortion curve (fig. 1) had many points of inflection. Consequently, the curve was divided into two segments—0 to 70 mm. and 70 to 140 mm.—to obtain enough accurate polynomials. An equation for each of the two segments of the total curve was determined by using radii and distortion data for 15 and 30 points, respectively, as electronic computer input data.

Adequate results based on the reliability of the input information were obtained by use of the polynomial curve fitting technique. Although lens distortion data were available for only 15 discrete radii at intervals of 10 mm. on the plate, a smooth curve was plotted through these points (fig. 1), and it was accepted as the actual distortion curve. Enough distortion data for specific radii were selected from the curve and used as input data for the curve fitting electronic computer program. All computed distortion results based on the computer curve fell within less than 0.5 micron of the plotted curve. This technique of radial lens distortion compensation is well within the accuracy tolerances of ±2 microns given by the manufacturer.

Relative orientation

Relative orientation may be defined as reconstruction of the geometric conditions existing between pairs of photographs at the moment of exposure (*12*). Relative orientation is the determination of three rotational (omega, phi, kappa) and two of three translational (bx, by, bz) elements that define the attitude and positions of one photograph to another, providing there was sufficient common area of overlap in line of flight.

The method of relative orientation in the evaluation reported here depends upon enforcing a geometric condition where the photographic image, perspective center, and the object in the stereoscopic model are on a straight line (colinear). (Complete details on the mathematical formulation and derivations of the relative orientation and other parts of the analytic system are explained in reference *1*.) For a given stereoscopic model, there are many lines (a pair for each image). But, because of different errors, it was impossible to enforce all colinear conditions completely. As the left-hand photograph of a pair is assumed to have no tilt in this method of analytic relative orientation, small corrections were made to the measured x and y coordinates of the right-hand photograph in a least squares manner. Each stereoscopic model was oriented independently; 12 image points were used, three computer iterations were required for completion of the relative orientation. The third and usually last iteration determined the x, y, and z coordinates for each point in the stereoscopic model and also orientation data that later were used in the assembly of independently oriented stereoscopic models to form a strip. The lack of intersection of pairs of lines (corresponding rays) for all the image points in each stereoscopic model were printed out as residual y parallaxes. These residuals were reviewed at the completion of the orientation. Substitutions were made for points having unusually large residuals, and a new orientation was performed.

Table 2.—Scale factors used to compensate for film deformation

Photographic plate	Compensation factor for axes—	
	x	y
1-40	0.999646	0.999646
1-41	0.999602	0.999611
1-42	0.999690	0.999690
1-43	0.999717	0.999788
1-44	0.999673	0.999805
1-45	0.999712	0.999797
1-46	0.999699	0.999814
1-47	0.999646	0.999735
1-48	0.999717	0.999761
1-49	0.999673	0.999766
1-50	0.999602	0.999797

This procedure was continued until no residuals larger than 25 microns remained at any given point. Table 3 shows the average absolute residual y parallax remaining at each of the 12 points in each of the stereoscopic models that were oriented. Residual y parallaxes were as large as 50 microns for some points in the computed stereoscopic models. These larger residuals were primarily the result of errors introduced by the incorrect position of the drilled holes, but the inaccuracy attached to measuring the holes and the method of film deformation compensation also contributed to the large residual determinations. The method of independent relative orientation of models employed is dependent upon the intersection of only two rays (lines) from the respective photographs. Thus, no check is possible to determine the accuracy of computed elevations of points. Errors in x parallax are reflected as errors in the elevation of points on the ground. The elevations are computed in the final step of the analytic system of aerial triangulation.

Table 3.—Average absolute residual y parallax from relative orientation

Model	y parallax residual
Number	*Microns*
1	5.9
2	8.8
3	9.6
4	5.0
5	6.0
6	8.2
7	9.1
8	7.7
9	7.7
10	9.7

Table 4.—Scale factors for cantilever assembly computations

Models	Scale factors
1 and 2	0.91743098
2 and 3	0.96043167 0.95981338
3 and 4	0.95736429
4 and 5	0.91459298 0.91491979
5 and 6	0.95755554 0.95778659
6 and 7	0.91712976
7 and 8	0.91668318 0.91630765
8 and 9	0.91052712 0.91084330
9 and 10	0.90574928

Table 5.—Computed strip coordinates of points in triple overlap areas

Point	Computed strip coordinates		
Number	x	y	z
1-40-B	0.98092 0.98092	0.22099 0.22100	−1.60790 −1.60809
1-42-B	0.94122 0.94121	0.71363 0.71364	−1.59301 −1.59322
1-42-L	1.93575 1.93575	0.60043 0.60049	−1.49853 −1.49885
1-42-K	1.94781 1.94780	0.14704 0.14712	−1.57810 −1.57766
1-42-D	1.93855 1.93855	−0.49565 −0.49581	−1.54782 −1.54813
1-42-G	2.88542 2.88542	−0.17437 −0.17442	−1.54318 −1.54327
1-42-J	2.92091 2.92092	0.19175 0.19172	−1.52242 −1.52299
1-44-F	3.81770 3.81770	0.81302 0.81318	−1.51911 −1.51931
42-2	3.82037 3.82037	−0.14599 −0.14601	−1.53610 −1.53600
1-44-D	3.80635 3.80636	−0.64514 −0.64507	−1.53752 −1.53737
1-44-K	4.78011 4.78013	0.59229 0.59277	−1.49192 −1.49278
1-44-G	4.79534 4.79537	−0.01733 −0.01719	−1.51885 −1.51975
41-2	5.67728 5.67728	0.11931 0.11932	−1.49878 −1.49888
1-48-F	7.53923 7.53923	−0.20051 −0.20049	−1.53959 −1.53961
38-1	7.53990 7.53990	0.59161 0.59178	−1.51816 −1.51848
1-50-E	8.45788 8.45788	0.48609 0.48586	−1.48191 −1.48140
1-50-A	8.46795 8.46795	−0.11971 −0.11996	−1.53105 −1.53033

Table 6.—Summary of errors for five third degree adjustments

Adjustment	Control points		Test points		Errors for third degree adjustments by type of error for each coordinate											
	Horizontal	Vertical	Horizontal	Vertical	RMSE			Maximum			Minimum			Algebraic mean		
					X	*Y*	*Z*	*X*	*Y*	*Z*	*X*	*Y*	*Z*	*X*	*Y*	*Z*
CONTROL POINTS																
Number	*Number*	*Number*	*Number*	*Number*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1	4	12	-	-	0.05	0.32	0.59	−0.08	−0.49	1.28	−0.01	−0.09	0.00	−0.01	0.00	0.15
2	5	18	-	-	0.09	0.34	0.09	−0.13	−0.56	−1.93	0.02	0.08	−0.03	0.01	−0.12	−0.26
3	4	13	-	-	0.05	0.14	0.75	0.08	0.22	−1.78	0.00	0.02	0.25	0.01	0.30	−0.12
4	4	10	-	-	0.00	0.04	0.34	0.00	−0.19	−0.91	0.00	0.01	0.00	0.00	−0.09	−0.19
5	4	7	-	-	0.10	0.18	0.00	±0.13	−0.27	0.00	−0.02	−0.05	0.00	−0.02	−0.05	0.00
TEST POINTS																
1	4	12	10	40	0.60	0.59	1.52	0.95	1.21	3.60	0.14	−0.01	0.12	−0.07	0.20	0.71
2	5	18	9	34	0.61	0.42	1.42	0.86	0.80	3.18	−0.02	−0.01	0.02	0.06	0.25	0.29
3	4	13	10	39	0.54	0.52	1.67	0.91	−0.97	3.94	0.04	0.08	−0.05	−0.11	−0.15	0.19
4	4	10	10	42	0.55	0.54	1.95	−0.96	−0.93	−4.63	0.02	0.06	0.01	−0.12	−0.09	0.05
5	4	7	10	45	0.69	0.69	1.89	−1.21	−1.22	4.61	−0.14	−0.08	−0.10	−0.48	−0.17	0.39

Table 7.—Summary of errors for five second degree adjustments

Adjustment	Control points		Test points		Errors for second degree adjustments by type of error for each coordinate											
	Horizontal	Vertical	Horizontal	Vertical	RMSE			Maximum			Minimum			Algebraic mean		
					X	*Y*	*Z*	*X*	*Y*	*Z*	*X*	*Y*	*Z*	*X*	*Y*	*Z*
CONTROL POINTS																
Number	*Number*	*Number*	*Number*	*Number*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1A	4	12	-	-	0.93	0.10	0.92	1.40	0.44	1.85	0.26	±0.05	0.10	0.00	0.00	0.02
2A	5	18	-	-	0.97	0.33	1.12	1.57	0.45	2.09	0.30	0.14	0.02	−0.01	−0.01	0.00
3A	4	13	-	-	0.41	0.12	1.25	−0.64	−0.17	2.02	−0.05	±0.01	0.07	0.00	−0.08	0.00
4A	4	10	-	-	0.51	0.10	1.21	−0.79	0.16	−1.94	−0.03	0.00	−0.39	0.29	0.00	−0.03
5A	4	7	-	-	0.81	0.23	0.84	1.20	0.38	−1.37	−0.22	0.02	0.17	−0.11	−0.11	−0.01
TEST POINTS																
1A	4	12	10	40	1.26	0.69	1.54	2.01	1.27	4.06	0.35	−0.16	−0.01	0.22	0.55	0.80
2A	5	18	9	34	1.17	0.77	1.51	2.21	1.26	3.89	0.05	0.07	0.01	0.32	0.48	0.27
3A	4	13	10	39	1.71	0.68	1.44	2.86	1.06	3.84	−0.15	−0.19	−0.03	0.58	0.10	0.27
4A	4	10	10	42	1.62	0.60	1.38	2.56	0.94	3.67	−0.36	−0.01	−0.18	0.96	0.19	−0.20
5A	4	7	10	45	1.61	0.61	1.46	−2.55	−1.47	3.84	0.43	0.09	−0.16	−1.17	−0.20	0.33

Cantilever Assembly

After the successive completion of the relative orientation for each of the stereoscopic models, the individual models were tied together into a continuous strip. This assembly of models is accomplished by successive mathematical transformation of the model coordinates of each model in the strip. The three transformations, which were performed in order, were rotation, scaling, and translation. The strip coordinates, which are analogous to those obtained from an analog triangulation instrument, were obtained from these mathematical computations. The first stereoscopic model in the strip was arbitrarily considered to be at the desired scale and its coordinates in the proper system. Therefore, the mathematical transformations were performed only on the second and succeeding models. Scaling in this cantilever system was accomplished by comparing slope distances between two image points, which occur in adjacent stereoscopic models (the common overlap area of three photographs), and making an adjustment by a scale factor to the model being attached to the strip.

Because of the limited storage capacity of the IBM 1401 computer used in the investigation reported here, only 10 points per stereoscopic model could be accommodated in the cantilever assembly program. For this investigation, however, 10 points per model were considered enough. Table 4 contains a listing of scale correlation factors computed from points occurring in the triple overlap area. Wherever enough points were available, two scale factors were computed and the average value used. Scale factors should be in reasonably close agreement. Points causing the anomalies in scale factors were discarded, and substitute points used to recompute the strip coordinates.

The strip coordinates of points occurring in triple overlap areas, which are listed in table 5, were computed by using data derived from the independently oriented stereoscopic models of the strip. Averages of two sets of strip coordinates for each point were used; they were considered the most acceptable for the point. There was slightly larger disparity in computed strip elevations than in the horizontal strip coordinates.

Adjustments

The adjustment of cantilever strip coordinates is the last computational step; this computation provides the *X*, *Y*, and *Z* ground coordinates (*13*). The adjustment is fully applicable to strip coordinates derived from either analog optical train bridging instruments or to coordinates derived from measurements made with monocomparators. The method of adjustment used, attempts to correct for curvature of the strip (azimuth), twist or cross tilt, *BZ* fall off, scale change along the *x* and *y* axes of the strip, and local tilt of the strip in the *x* and *y* directions. Cumulative errors in a strip tend to be systematic and can be corrected by use of polynomial adjustments. Both second and third degree polynomial adjustments were applied to determine their effectiveness.

To accomplish the vertical and horizontal adjustment of the strip coordinates, data listed in the following statements are required

Table 8.—Errors in horizontal coordinates from second degree adjustments

Point	Errors in horizontal coordinates from second degree adjustments									
	1A		2A		3A		4A		5A	
	X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
Number	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
45-1	1.89	−0.16	2.21	−0.54	1.72	−0.43	1.92	−0.32	[1] −0.22	0.02
SW-45	[1] 0.26	−0.05	[1] 0.60	−0.35	[1] 0.25	−0.17	[1] 0.59	−0.13	−1.18	0.17
SW-44	−1.04	0.25	−0.58	0.07	−0.60	−0.19	[1] −0.79	0.16	−2.04	0.26
SW-43	−1.54	0.30	[1] 0.90	0.17	[1] −0.64	0.01	−0.80	−0.01	−2.29	−0.19
42-1	−2.01	0.30	−1.47	0.24	−1.11	0.35	−1.22	0.72	2.55	0.09
42-2	−1.39	0.83	−0.81	0.91	−0.15	−0.92	−0.36	0.74	−1.64	0.43
SW-41	[1] −0.87	0.05	[1] −0.30	0.14	[1] 0.44	−0.17	[1] 0.72	−0.04	[1] −1.05	−0.38
41-2	−0.35	0.43	0.14	0.54	1.12	0.52	0.87	0.29	−0.44	−0.18
40-1	−0.48	0.39	0.05	−0.23	0.92	−0.34	0.68	−0.56	0.67	−0.94
38-1	0.70	1.27	1.02	1.26	2.20	1.06	1.94	0.94	0.43	0.22
39-1	1.29	0.95	1.71	1.00	2.86	0.93	2.56	0.74	[1] 1.20	0.13
SW-38	[1] 1.40	−0.44	[1] 1.57	−0.42	−2.69	−0.72	2.51	−0.81	0.81	−1.47
SW-37	[1] 0.80	0.44	[1] −0.98	0.45	[1] 0.05	−0.01	[1] −0.03	0.00	−2.11	−0.52
BMC 131	0.71	0.97	0.63	1.11	1.55	0.76	1.54	0.68	[1] −0.38	−0.22

[1] Horizontal control points used to adjust strips.

as inputs: (1) A card containing the number of vertical and the number of horizontal ground control points used in the adjustments; (2) one card containing the *x* and *y* cantilever strip coordinates of a point near the center of the first stereoscopic model and another point near the center of the last model in the strip; (3) the strip *x*, *y*, and *z* coordinates of all points to be used as a basis for the adjustment for which the ground *X*, *Y*, and *Z* coordinates are known; (4) the horizontal and vertical ground control data for the points used in item 3; and (5) the cantilever strip coordinates needed to establish supplemental control. Three horizontal and five vertical control points are the minimum number needed to make a second degree adjustment, and four horizontal and seven vertical control points are needed to make a third degree adjustment.

Computed Ground Coordinates

The errors summarized for the computed ground coordinates using second and third degree cantilever strip adjustments are shown in tables 6 and 7. The horizontal and vertical errors for ground coordinates computed for 55 separate points in 10 trial adjustments are listed in tables 1, 8, and 9. The distribution of ground control used in each of the adjustments is shown in figure 5. The third degree horizontal adjustments were the most accurate. In table 6 the RMSE are given for the horizontal coordinates of the third degree adjustment. RMSE ranged from 0.42 and 0.69 of a foot and no significant difference was shown between the *X* and *Y* coordinates. The use of five rather than four horizontal control points did not significantly increase the accuracy of the computed horizontal coordinates.

The RMSE for the horizontal coordinates of second degree adjustments are shown in table 7; these RMSE ranged from 0.60 to 1.71 of a foot. The magnitude of the error in the *X* coordinates was about twice that for the *Y* coordinates.

The errors in the computed *Y* coordinates in both second and third degree adjustments were about the same. Errors in the computed *X* coordinates of the third degree adjustment were one-half the size of those for the second degree adjustments. Test point 45–1 tended to have slightly more error in horizontal coordinates in 8 of the 10 adjustments because of its position outside the confines of ground control point SW–45, located near the beginning of the flight strip.

A comparison of the vertical RMSE in tables 6 and 7 for the second and third degree adjustments shows no significant difference in accuracy for the first three sets of trials. Twelve or more vertical control points were used to adjust these flight strips. Comparison of the second degree adjustments, however, showed a significant improvement in accuracy over the third degree in the last two sets of adjustments, where vertical control points were not as dense.

An increase from 7 to 18 vertical control points had practically no effect on the RMSE of the second degree adjustments, but it did significantly reduce the RMSE of the third degree adjustments. The largest vertical errors were for test points located in areas where no vertical control points were present in their vicinity (adjustments 4, 4A, 5, 5A). The effects of density and distribution of vertical control points on ultimate accuracy are difficult to analyze. It is likely that some of the holes drilled for ground control points were not in stereoscopic correspondence with the ground surface. Only those control points observed to float or to dig in the Kelsh instrument were eliminated as control points for making the strip adjustments.

The algebraic mean errors listed in tables 6 and 7 for elevations of test points are positive for all trial adjustments except adjustment 4A. The positive sign of these mean errors tends to confirm the stereoscopic observations made of the drilled holes and the existence of systematic errors.

Summary

The author recognizes the fact that the method used to compensate for film deformation is inadequate, but it is the most suitable method that can be used for cameras equipped with only four side fiducial marks. It is, however, impossible to prove the amount of error in ground coordinates caused by film deformation. The estar base film is considerably more stable dimensionally than film bases previously available. At least three possible alternatives could be used for a more effective treatment of the film deformation problem. First, existing cameras could be equipped with four additional fiducial marks so that a more effective mathematical restoration of displaced images can be made. A computer program is available to accomplish this. Second, a network of small crosses (reseau) could be used in the focal plane of the camera. The crosses are recorded on the negative film at the instant of exposure. Displacement of images caused by film distortion or lack of film flatness at the instant of exposure could be compensated for by comparing the measured position of the reseau crosses with their calibrated position. Third, a glass plate aerial camera could be used; this would eliminate the need for compensation caused by film deformation.

When measuring plate coordinates with a monocomparator, some form of point transfer and marking technique is a practical necessity. The PUG point transfer device was used for the project discussed here. Primarily because of inexperience with the instrument, some error was introduced. A divergence of opinion exists at the present time regarding the inherent accuracy of several point transfer and marking devices. Research and evaluation of these techniques is being done by private firms, universities, and governmental agencies. Ground control and other points that are premarked with photographic targets can be reliably identified and measured without need for point transfer and marking.

The relative size of the measuring mark to that of the drilled hole appears to be a significant factor in the degree of accuracy of measurement. A 20-micron measuring mark, a fiducial hole having a diameter of 250 microns, or a drilled hole 60 microns in diameter are not desirable combinations for optimum accuracy of measurement. Better accuracy could be obtained by use of a larger measuring mark. Some doubt has also been expressed about the adequacy of drilled holes that are 60 microns in diameter for use in map compilation with the Kelsh instrument. At a 5:1 projection ratio, a significant number of holes could not be seen in the stereoscopic models. A drilled hole of at least 100 microns in diameter is needed for use with the Kelsh instrument.

Basic ground control surveys on which aerial analytic triangulation is based should be of second order accuracy (1/10,000 closing error in position) or better, so results of the triangulation will not be degraded because of inferior control. When this control is used, results of the triangulation as to accuracy can be more realistically evaluated. Results of the aerial analytic triangulation cannot be expected to be any better than the ground control survey on which the triangulation is based.

The polynomial curve fitting technique used to determine an equation for the radial lens distortion curve is considered an acceptable method. It is particularly adapted for use in determining the equations of curves that are

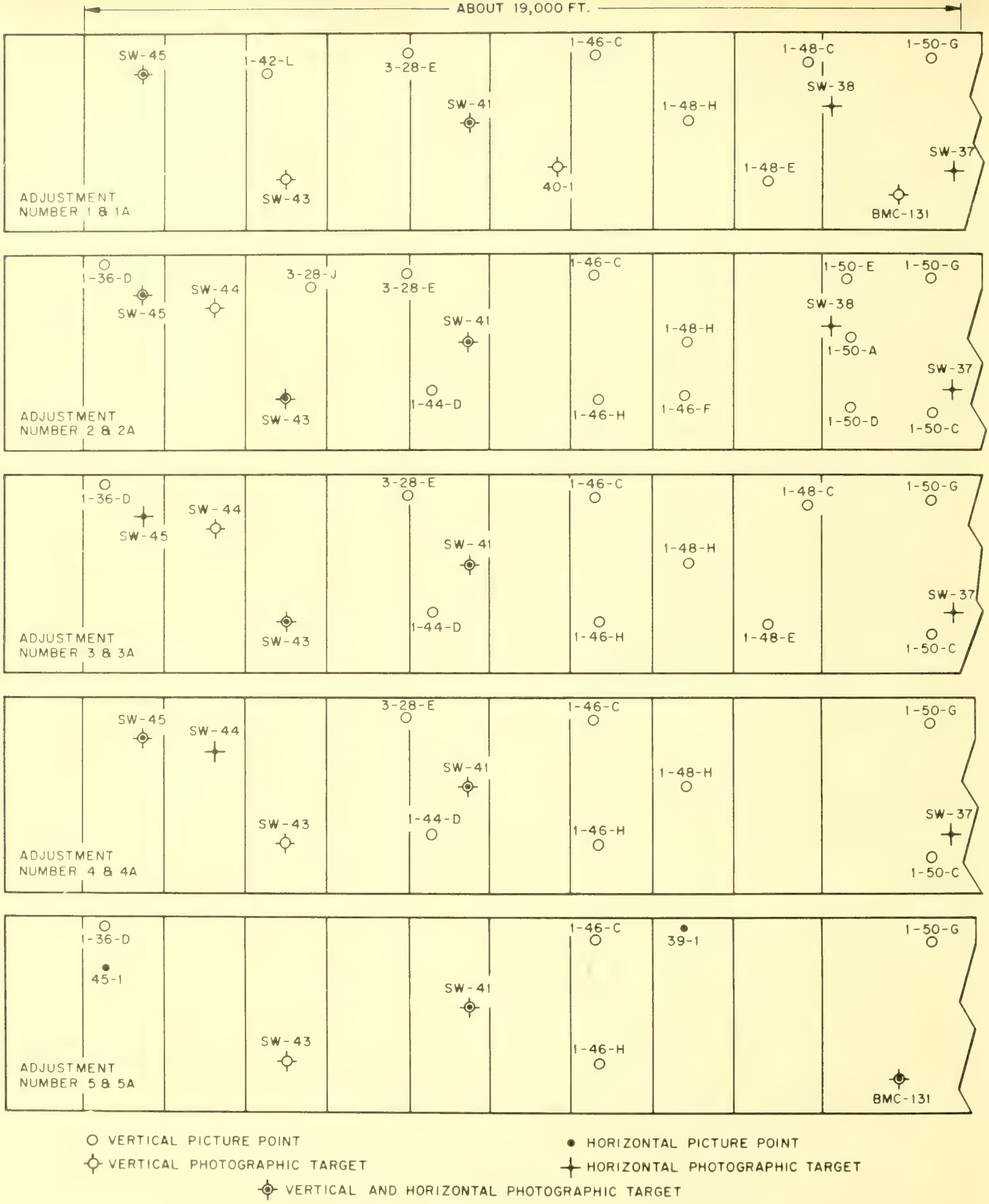

Figure 5.—Distribution of ground control used in different adjustments.

smooth and have only a few points of inflection. Corrections made to the x and y measured coordinates, based on the computed curve, were within the accuracy tolerances for the manufacturer's determination of the lens distortion values.

Corrections for atmospheric refraction and adjustments for earth curvature were not considered in the investigation reported here. Displacement of photographic images by atmospheric refraction and earth curvature are negligible because of the relatively low flight height from which the photographs were taken and the short length of the flight strip. For bridging photographs taken from higher flight heights and for longer flight strips, appropriate adjustments should be made to compensate for atmospheric refraction and earth curvature (*14*, *15*).

When small capacity computers are used for aerial analytic triangulation, considerable segmentation of the computer programs and excessive card handling is required. Computers that have larger storage capacity and more speed must be used if the analytic operations are to be performed more efficiently.

Use of mapping scale photographs (1:6,000) for analytical bridging does not seem to be the most accurate or economical approach for obtaining supplemental ground control data for large scale topographic mapping. Use of the larger scale photographs requires measurement of x and y for a larger number of stereoscopic models for a given bridged distance. The large number of intermodel ties needed tends to deform the bridged strip and thus offset the advantages gained from having a larger scale.

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Table 9.—Errors in vertical coordinates from second and third degree adjustments, 5 adjustments for each degree [1]

Point	Errors in vertical coordinates from the 5 adjustments									
	1	1A	2	2A	3	3A	4	4A	5	5A
Number	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1-40-B	−0.75	−1.36	−0.73	−1.67	−0.36	−0.85	−0.83	−1.84	−0.80	−2.09
1-42-B	−0.80	−0.73	−1.36	−1.00	−1.79	−1.27	−2.51	−1.26	−2.57	−1.26
1-36-D	1.61	1.00	0.71	[2] 0.62	0.70	−0.30	0.31	−0.29	[2] 0.00	−0.17
45-1	2.99	1.35	2.94	0.96	3.94	0.65	3.88	0.75	3.68	0.41
1-40-A	−0.50	−0.13	−1.43	−0.40	−1.22	−0.75	−3.02	−0.71	−3.17	−0.75
SW-45	[2] 0.59	[2] −0.12	0.23	[2] −0.45	0.47	−0.75	[2] 0.05	[2] −0.69	0.10	−0.91
45-2	3.60	2.67	3.18	3.40	2.83	3.17	2.13	3.17	2.10	3.05
1-42-L	−1.78	[2] −1.09	−2.19	−1.27	−3.05	−1.42	−3.92	−1.49	3.85	−1.46
1-42-K	[2] −0.05	0.29	[2] −0.07	−0.02	−0.60	−0.09	−1.32	−0.18	−1.23	−0.21
1-42-D	−0.12	−0.06	−0.19	−0.44	0.57	−0.50	0.05	−0.61	0.21	−0.70
1-40-C	1.36	−0.35	2.18	0.73	3.77	−0.80	3.68	−0.79	3.83	−1.25
SW-44	−1.29	[2] −1.23	[2] −0.54	[2] −1.50	[2] −1.78	[2] −1.68	2.47	−1.70	−2.44	−1.83
1-42-C	−2.52	−1.99	−3.01	−2.20	−3.78	−2.41	−4.63	−2.44	−4.61	−2.48
SW-43	[2] −0.03	[2] −0.10	[2] −0.03	[2] −0.52	[2] 0.34	[2] −0.57	[2] −0.18	[2] −0.72	[2] 0.00	[2] −0.74
1-42-G	−0.54	1.13	0.23	0.81	−0.22	0.76	−0.87	0.57	−0.75	0.75
1-42-J	3.44	4.06	0.16	3.89	2.46	3.84	1.77	3.67	1.87	3.84
3-28-J	−1.03	−0.16	[2] −1.38	−0.28	−2.46	−0.38	−3.33	−0.49	−3.27	−0.41
1-42-H	0.20	1.03	−0.06	0.93	−1.12	0.87	−1.91	0.74	−1.84	0.88
1-42-F	1.00	1.18	0.80	0.72	1.02	0.67	0.47	0.47	0.64	0.57
42-1	−0.66	−0.01	0.94	−0.24	−1.67	0.31	−2.35	−0.45	−2.24	−0.40
1-44-F	1.95	2.38	2.05	2.36	1.23	2.42	0.80	−2.26	0.78	2.40
42-2	1.39	1.97	0.02	1.64	−0.50	1.62	0.05	1.36	0.13	1.73
1-44-D	0.92	2.58	[2] −0.29	[2] 2.09	[2] 0.91	[2] 2.02	[2] −0.44	[2] 1.71	0.57	2.21
1-44-A	0.54	1.19	0.29	0.97	−0.40	0.95	−0.99	0.75	−0.93	0.98
3-28-F	1.62	2.23	1.47	2.09	0.68	2.08	0.12	1.90	0.16	2.10
1-44-B	1.88	2.01	1.13	1.92	0.14	1.90	−0.53	1.76	−0.49	1.90
3-28-E	1.28	[2] 1.85	[2] 1.29	[2] 1.81	[2] 0.37	[2] 1.84	[2] −0.16	[2] 1.69	0.16	1.83
1-44-K	1.85	1.75	2.11	1.67	1.79	1.81	1.81	1.59	1.72	1.81
1-44-G	0.89	1.05	0.67	0.73	0.35	0.70	0.27	0.46	0.30	0.94
3-26-D	2.33	2.38	1.62	2.35	2.13	2.48	2.02	2.29	1.94	2.45
SW-41	1.04	1.47	[2] −0.73	[2] 1.16	[2] −0.25	[2] 1.16	[2] −0.07	[2] 0.87	[2] 0.00	[2] 1.29
41-2	0.69	0.34	0.68	0.01	0.73	0.12	1.05	−0.19	1.02	0.29
3-26-E	1.86	0.39	2.24	1.29	2.25	1.48	2.58	1.25	2.43	1.48
40-1	[2] −0.20	[2] 0.13	−0.98	−0.52	−1.24	−0.61	−1.32	−1.03	1.07	−0.16
1-46-E	−0.72	−1.78	−0.21	−1.90	0.42	−1.57	−1.27	−1.79	0.99	−0.68
1-48-H	[2] −0.47	[2] −1.01	[2] −0.79	[2] −1.61	[2] −0.50	−1.55	[2] 0.00	[2] −1.94	0.26	−1.19
1-46-F	0.31	0.17	[2] −0.61	[2] −0.77	0.60	−0.92	−0.36	−1.44	0.12	−0.20
1-46-C	[2] −0.85	[2] −1.47	[2] −0.44	[2] −1.58	[2] −0.29	[2] −1.36	−0.16	[2] −1.59	[2] 0.00	[2] −1.37
1-46-H	0.78	0.97	[2] −0.12	[2] 0.20	[2] −0.31	[2] 0.07	[2] −0.28	[2] −0.39	[2] 0.00	[2] 0.64
1-46-D	0.56	−1.25	−0.45	−1.58	−0.11	−1.40	0.47	−1.71	0.40	−1.27
1-48-F	0.74	0.11	0.35	−0.60	−0.83	−0.50	1.46	−0.90	1.76	−0.13
1-48-E	[2] 0.20	[2] −0.12	−0.84	−1.26	[2] −0.72	[2] −1.45	−0.45	−2.02	0.40	−0.58
1-50-E	1.19	0.71	[2] 0.99	[2] 0.33	1.79	0.80	2.70	−0.59	2.58	0.60
1-50-A	1.35	1.05	[2] −0.72	[2] 0.25	1.19	0.42	1.74	0.05	2.23	0.73
1-48-C	[2] 0.25	−0.54	0.25	−0.91	[2] 1.07	[2] −0.51	2.02	−0.76	1.89	−0.62
1-50-D	0.13	0.05	[2] −1.03	[2] −1.30	−1.00	−1.52	−0.91	−2.00	0.41	−0.54
1-50-G	[2] 0.00	[2] 0.72	[2] −1.93	[2] 0.23	[2] −0.55	[2] 0.80	[2] −0.03	[2] 0.62	[2] 0.00	[2] 0.48
1-50-C	2.33	2.93	[2] 0.86	[2] 1.30	[2] 0.49	[2] 1.01	[2] 0.04	[2] 0.41	2.08	2.14
BMC-131	[2] −0.01	[2] 0.26	−1.38	−1.32	−0.59	−1.61	−1.82	−2.23	[2] 0.00	[2] −0.48
1-50-B	0.84	0.98	−0.14	−0.05	−0.05	−0.03	0.30	−0.39	1.21	0.52
1-50-F	0.36	0.47	−0.22	0.03	0.39	0.55	1.12	0.36	1.08	0.29

[1] Second degree adjustment is indicated by the letter A after the number.

[2] Vertical control points used to adjust strip elevations.

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Linear Shrinkage Test

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

**Reported by GEORGE W. RING III,
Highway Research Engineer,
Structures and Applied Mechanics Division**

Introduction

RESULTS OF a study of the linear shrinkage test for soils are presented in this article. In another article, *Shrink-Swell Potential of Soils* (*1*)[1] linear shrinkage was shown to be closely related to the shrink-swell potential of a soil. The purpose of the study reported herein was to develop a modified linear shrinkage test procedure whose results would correlate even better with shrink-swell potential. Although a procedure for a better correlation was not obtained from the study, results showed how certain variations in test procedure affect the linear shrinkage of soils. The research information has been assembled for this article primarily for engineers interested in the linear shrinkage test.

Included is a brief review of three published linear shrinkage test procedures: Bureau of Public Roads test data, showing the variables that affect the linear shrinkage, including a quantitative amount of the variation; and a comparison of two correlations developed with shrink-swell potential. The linear shrinkage test procedure used for the *Shrink-Swell Potential of Soils* study is printed at the end of this article.

There are several published methods for conducting linear shrinkage tests. In one, *Determining the Shrinkage Factors of Soils*, AASHO T 92-60 (*2*), the linear shrinkage is calculated from the volumetric shrinkage of a circular disk dried from the field moisture equivalent. However, the field moisture equivalent is seldom determined in highway laboratories. Also, preliminary investigations in this study showed that for specimens molded at the field moisture equivalent, the volumetric shrinkage is very dependent on the molding moisture content. For these reasons, the AASHO method was not considered further.

In another method, the Texas Highway Department's linear shrinkage test—bar method—(*3*), the soil is molded into a ¾- by ¾- by 5-inch bar when the soil is just wet enough to flow into and close a groove of its own accord. Linear shrinkage is the decrease in length of the dried bar expressed as a percentage of its original length.

A third method, similar to the Texas method, has been proposed by Hondros (*4*). In this method the soil is mixed with enough water to provide a free-water surface and placed into a mold 10 inches long. This mold is shaped like one-half of a tube that has a radius of one-half inch. The Hondros test procedure has been used primarily to determine linear shrinkage of the fines in lateritic gravels of Australia.

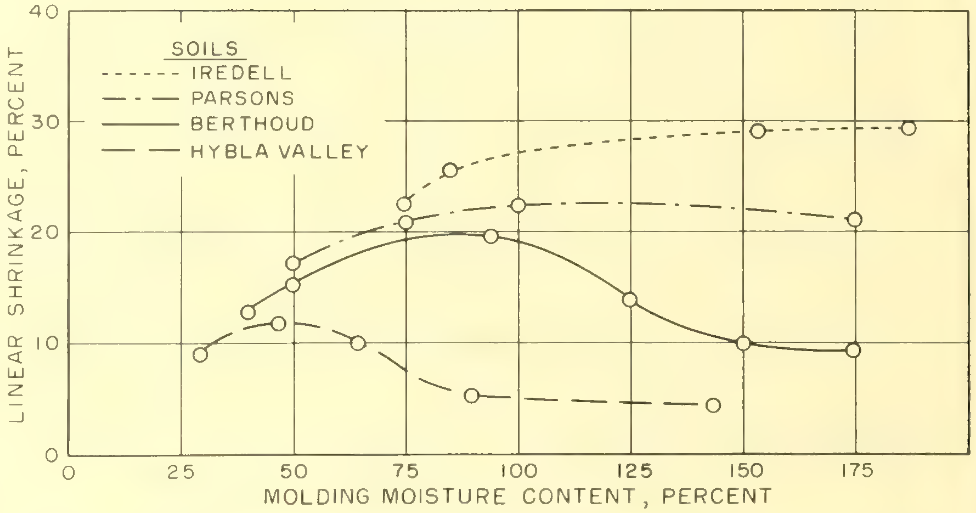

Figure 1.—Relation of linear shrinkage to molding moisture content of four soils.

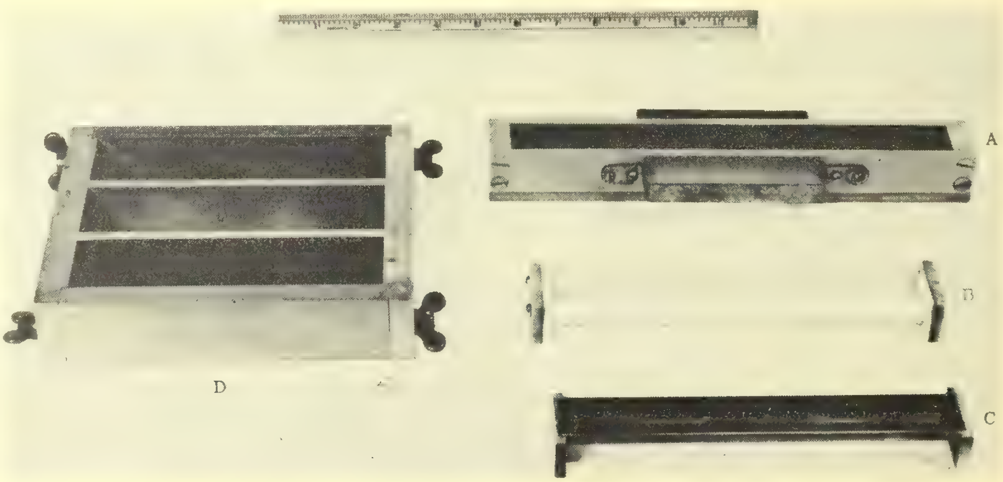

Figure 2.—Four types of linear shrinkage molds: A, steel and a square cross section; B, Teflon, semicircular; C, brass, semicircular; D, steel, square cross section.

From observations of linear shrinkage specimens molded at high moisture contents (1½ to 2 times the liquid limit) as in the method proposed by Hondros (*4*), it was noted that the specimens did not begin to shrink in the direction of their long dimension until enough water had evaporated for capillary forces to exceed the strength of the soil. The point at which shrinkage began, then, was not only a function of the soil's capacity to hold water, but also the soil's clay activity and the resisting strength of the soil structure. In hopes that results of a linear shrinkage test procedure based on such high molding moisture contents would be more closely related to shrink-swell potential, tests were made on several soils to determine how best to obtain the maximum linear shrinkage for each soil. Some of the same 12 soils examined in the shrink-swell potential study (*1*, table 3, p. 100) were used to determine the effect of the different variables on linear shrinkage. For most of the testing, specific soils were selected to obtain the probable maximum effect of a given variable on linear shrinkage. For example, a commercial grade halloysite is known to be difficult to mix. Therefore, halloysite was used to determine the difference in linear shrinkage of test specimens prepared by hand mixing and mechanical mixing. Except where noted, the soils were mixed with distilled water. When one variable was studied, all the other variables were held constant. Variables investigated included molding moisture content, type of mold, mold lubricant, mixing methods, drying temperature, flocculation, and curing time of the mixture.

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[1] References identified by italic numbers in parentheses are listed on page [illegible]

Molding Moisture Content

Linear shrinkage of some soils is affected by the molding moisture content, as illustrated by the curve in figure 1. Molding moisture contents for each soil tested ranged from below the liquid limit to saturation, which was considered to have been reached when free water was visible. In general, as molding moisture content is increased, the linear shrinkage increases to a maximum and then decreases. At high molding moisture contents, the linear shrinkage of two of the soils (Berthoud and Hybla Valley) was sharply reduced because of separation of the sand and clay-size particles, the sand settled to the bottom of the mold and the clay left on the top of the specimen curled and cracked badly. Elimination of this problem by flocculation of the soil moisture is discussed subsequently.

Type of Mold

Linear shrinkage molds in current use vary in cross section, length, and material. Some types of these molds are shown in figure 2. Mold A of steel has a square cross section; mold B of Teflon is semicircular; mold C of brass is semicircular; and mold D of steel has a square cross section for three specimens. Results of tests made in the three single molds, A, B, and C, with one soil show that the type of mold has a slight but noticeable effect on linear shrinkage, as shown by figure 3. The most linear shrinkage occurred when the semicircular mold made of Teflon was used, probably because Teflon has less affinity for water than for brass or steel. The Teflon mold also had a smaller surface area than the steel mold having the square cross sections. Limited investigations indicated that mold length has little or no effect on linear shrinkage test results but that specimens in the longer molds tend to crack more. Specimens molded at lengths longer than 10 times their diameters are difficult to measure after they are dry because they crack into many pieces.

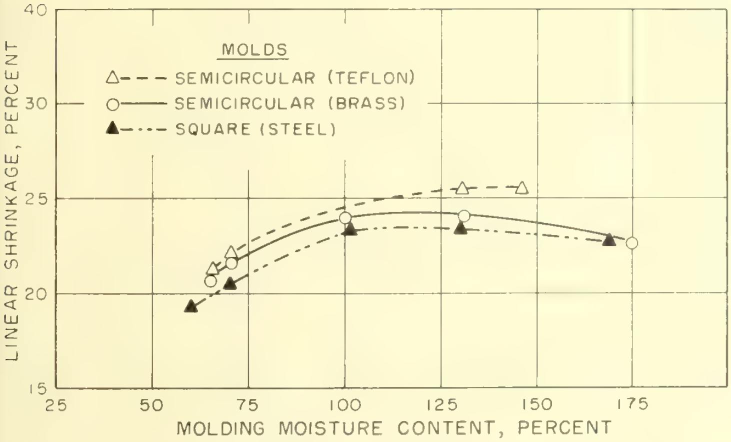

Figure 3.—Relation of type of mold to linear shrinkage of Winterset soil.

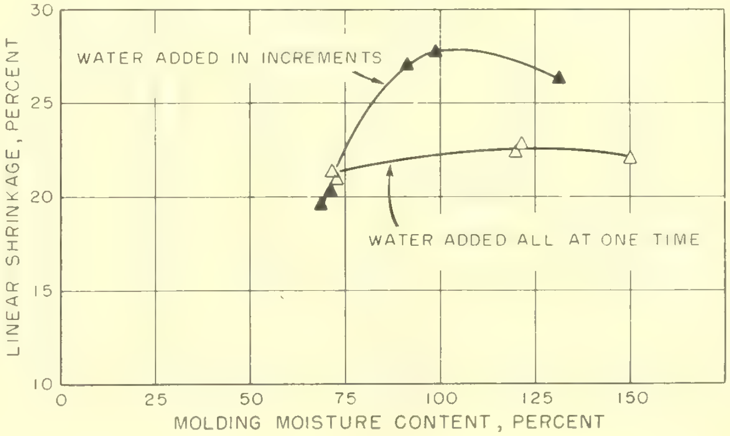

Figure 5.—Effect of mixing procedure on linear shrinkage of Winterset soil.

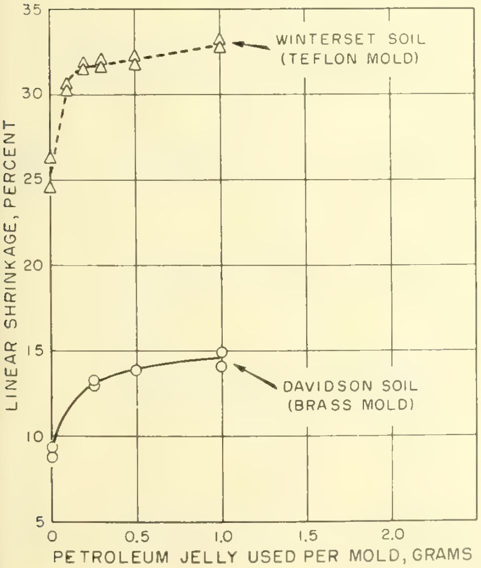

Figure 4.—Effect of amount of lubricant on mold on linear shrinkage of two soils.

Mold Lubricant

During drying, linear shrinkage specimens sometimes curl and crack, especially long specimens molded at high initial moisture contents. The broken and curled pieces are difficult to measure. Breaking and curling can be reduced by lubricating the mold. Petroleum jelly generally is used as a lubricant. Four other lubricants investigated in an attempt to find one more efficient than petroleum jelly were: Chassis grease, silicone release agent, Molycote type M, and Siliclad solution. None of the lubricants caused an increase in linear shrinkage or less cracking of the specimens than petroleum jelly, which is cheap and easy to obtain. Petroleum jelly was therefore used in all other tests.

It was determined that the linear shrinkage was dependent on the amount of lubricant. As shown in figure 4, the linear shrinkage of two soils increased considerably as the amount of petroleum jelly lubricant was increased from 0 to 0.25 gram per mold. Surface area of each of the molds was 16.5 square inches. A small, but consistent increase in shrinkage occurred when the amount of jelly used was increased over the initial 0.25 gram; however, 0.30 gram was selected as the proper amount of lubricant for all subsequent tests.

Mixing

Soil-water mixtures for linear shrinkage tests are generally prepared by hand mixing. As shown in figure 5, when all the dry soil was added to all the water at one time, less shrinkage occurred than when the water was gradually added to and thoroughly mixed into the soil.

When a milkshake machine was used to mix the soil and water, more linear shrinkage occurred than when the hand method was used for mixing. The effect of mixing methods on the linear shrinkage of a halloysite and Iredell clay is shown in figures 6 and 7. The halloysite required about 15 minutes of mixing with the milkshake machine to minimize the differences that might occur as a result of the mixing method, but the Iredell clay required only 5 minutes. The difference in the test results of duplicate tests is not readily understood.

Drying Temperature

Linear shrinkage specimens put immediately into a drying oven at 110° C. tend to

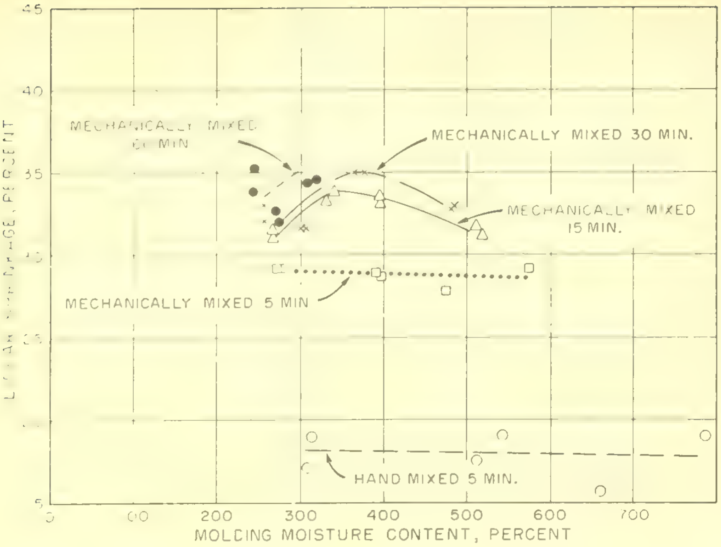

Figure 6.—Effect of mixing method on linear shrinkage of Halloysite soil.

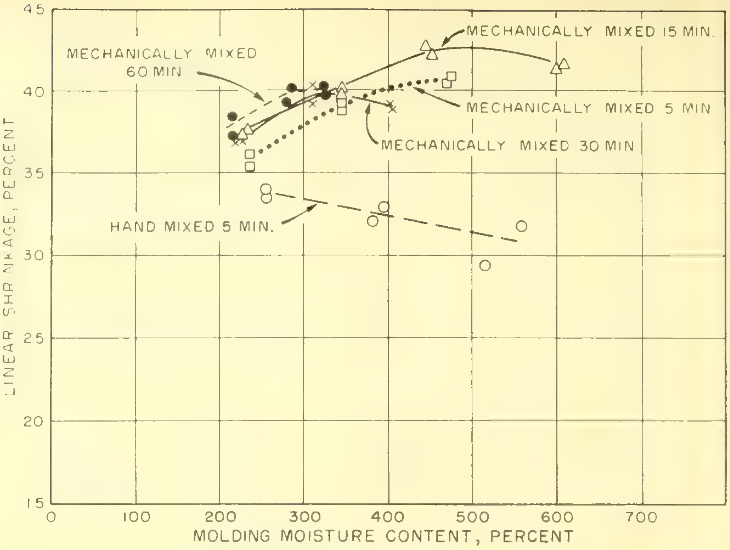

Figure 7.—Effect of mixing method on linear shrinkage of Iredell clay.

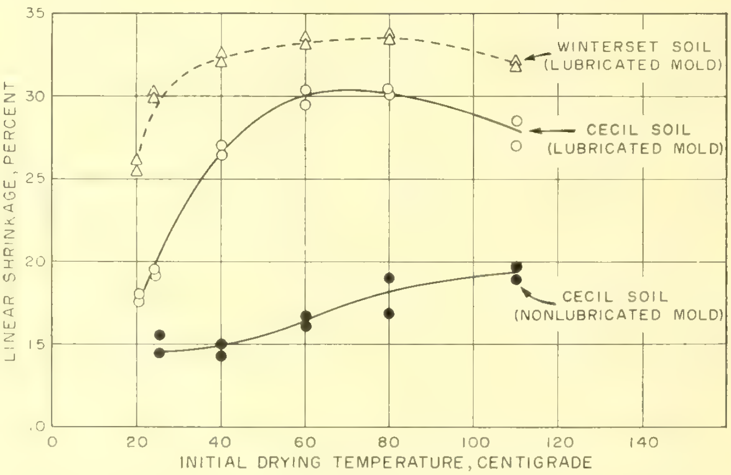

Figure 8.—Effect of drying temperature on linear shrinkage.

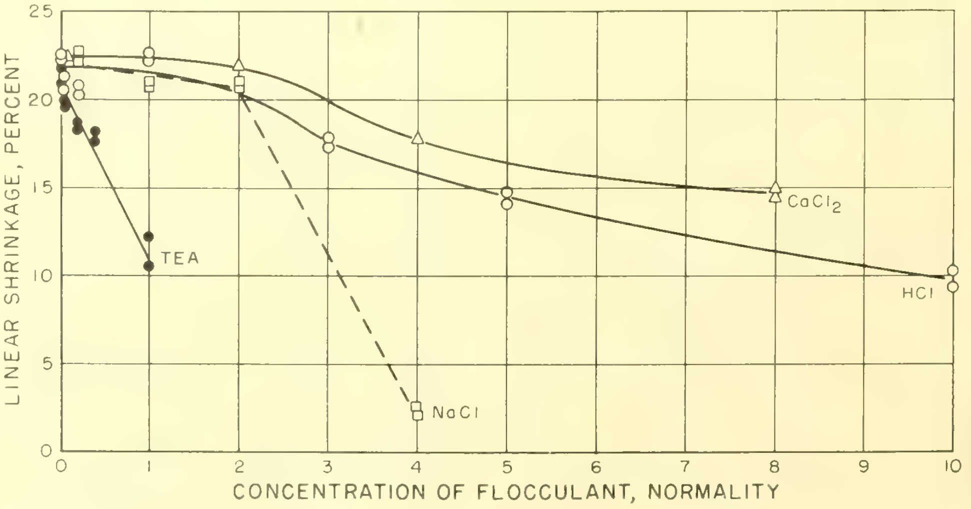

Figure 9.—Effect on linear shrinkage of Parsons soil when different concentrations of four flocculating agents were used.

crack more than those first dried at room temperature. Tests on two soils also indicate that for specimens in lubricated molds, the most linear shrinkage occurs when the initial drying temperature is between 60° and 80° C., as shown in figure 8. Results from tests made on one soil in unlubricated molds show that part of the change in linear shrinkage is caused by a change in the soil-water interaction at different temperatures. In the lubricated molds, other variations in linear shrinkage test results may have been the effect of different drying temperatures on the viscosity of the petroleum jelly. Although these tests indicate that 60° to 80° C. is the best temperature to use for oven drying, the linear shrinkage at 110° C. is not much less than at 60° to 80° C., and some laboratories may prefer to use the higher temperature so that the same oven can be used for other purposes.

Type of Liquid

As mentioned in the section on molding moisture content, the sand particles in linear shrinkage specimens molded at high moisture content tended to settle out and a thick layer of clay was left at the top. The result was a linear shrinkage specimen that had a crumbly sandy bottom and a badly cracked top so that corresponding reductions occurred in linear shrinkage. It was reasoned that if the soil mixtures were flocculated, the separation of the different soil particle sizes might not occur, and a uniform and easily measured specimen would be obtained.

Some salts and acids are good flocculants for soil-water mixtures. Accordingly, tests were made first to determine a good flocculating agent and then to determine whether flocculation with this agent affected the test results adversely.

A series of linear shrinkage tests was made on the Parsons soil using different normal concentrations of $CaCl_2$, NaCl, HCl, and triethylamine hydrochloride (TEA). Soil mix-

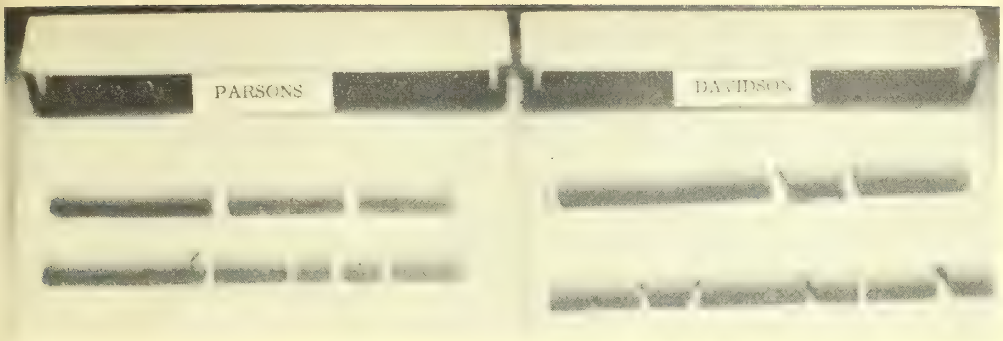

Figure 10.—Cracking shown for two soils mixed with (upper) and without a flocculating agent.

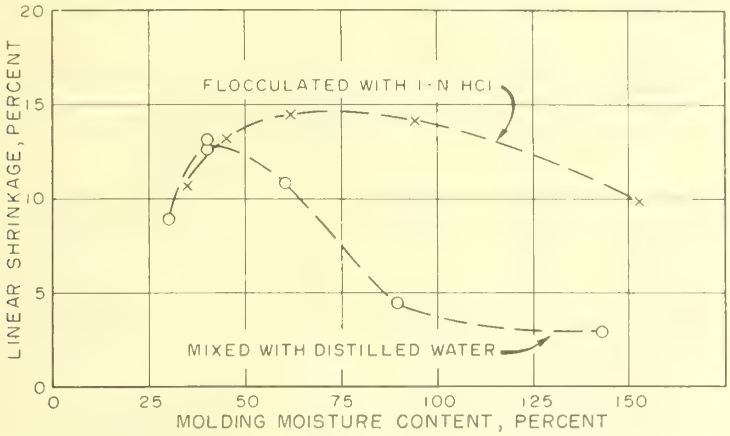

Figure 11.—Effect of flocculation on linear shrinkage of Hybla Valley clay.

Table 1.—Summary of variables that affected results of linear shrinkage tests

Variable and soil tested	Most change in linear shrinkage with change in variable	Maximum linear shrinkage	Change in linear shrinkage
Molding moisture content:	*Percent*	*Percent*	*Percent*
Winterset	5.3	22.5	+24
Hybla Valley	4.0	14.5	+28
Type of mold, steel, brass, and Teflon:			
Winterset	1.5	24.5	+6
Amount of lubricant:			
Davidson	4.7	13.9	+34
Winterset	7.3	32.0	+23
Mixing:			
Hand procedure:			
Winterset	5.4	27.6	+20
Milkshake vs. hand procedure:			
Halloysite	16.0	34.0	+47
Iredell	11.0	42.5	+26
Initial drying temperature:			
Cecil	12.7	30.3	+42
Winterset	7.5	33.5	+22
1-Normal solution of HCl vs. distilled water:			
Hybla Valley	10.3	13.8	+75
Portneuf	1.0	5.0	[1] −20
Winterset	2.0	18.0	[1] −6
Curing time:			
Halloysite	4.0	33.5	[2] −12
Iredell	1.0	43.4	[2] −2

[1] Flocculated specimens had less linear shrinkage than specimens prepared with distilled water.

[2] Cured samples had less linear shrinkage than uncured samples.

tures containing the calcium and sodium salts and the hydrochloric acid all showed a marked reduction in linear shrinkage at concentrations above 2-Normal, as shown in figure 9. The TEA was not judged suitable because of the reduction in linear shrinkage that occurred even at low concentrations. The two salt solutions were not suitable because the salt crystalization in the dried specimens produced rough specimens and made moisture measurements difficult. The HCl solution was deemed the most suitable flocculant and a 1-Normal solution was used on tests with the 11 other soils.

The HCl solution reduced specimen cracking for all soils tested and thereby simplified the process of making length measurements. Examples of the reduction in cracking for the Parsons and Davidson soils are shown in figure 10. Specimens shown at the top of the figure were mixed with the 1–N HCl solution and those at the bottom were mixed with distilled water. Preventing separation of the clay and sand sizes by flocculation also increased the linear shrinkages at high molding moisture contents, as shown in figure 11. Comparisons of flocculated specimens to those mixed with distilled water are shown in figure 12 for four other soils. For these soils, which contained few sand size particles, the flocculated specimens generally had slightly less linear shrinkages.

Curing Time

After being mixed, samples sometimes were permitted to cure, either purposely or because of delay caused by other laboratory operations, before they were placed in the shrinkage molds. Tests on two soils showed that curing time had a small effect on linear shrinkage. Figure 13 shows that the linear shrinkage

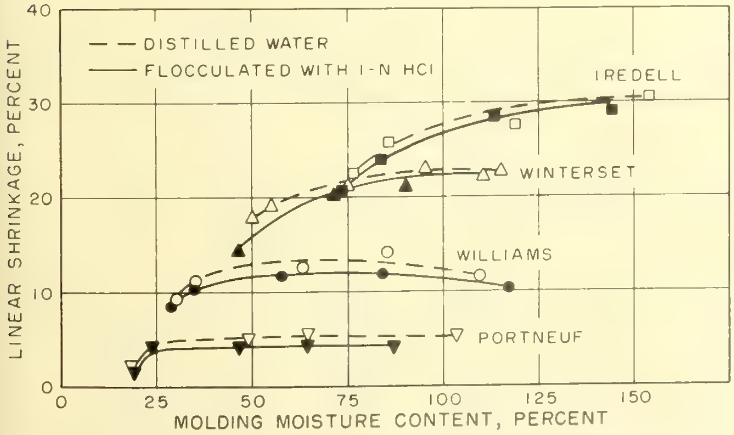

Figure 12.—Linear shrinkage of flocculated and nonflocculated soils.

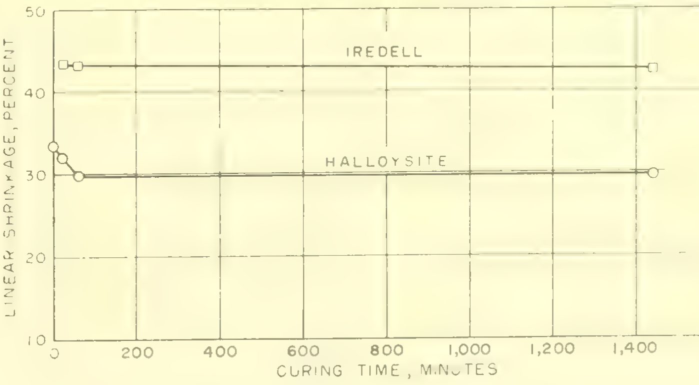

Figure 13.—Effect of curing time on linear shrinkage.

of halloysite was reduced at curing times of up to 60 minutes. The change in linear shrinkage of the Iredell clay in relation to curing time was hardly noticeable.

Summary of Study of Variables

All the variables investigated had an effect on the linear shrinkage of the soils tested. A summary of the variables and a quantitative measure of the amount of the variation is given in table 1. The amount of the variation was surprisingly large, especially for the mixing procedure, flocculation of sandy clays, amount of lubricant, and initial drying temperature. However, these variations were determined at high molding moisture contents and it is likely that specimens molded near the liquid limit would show less variation caused by mixing procedure and flocculation of sandy clays.

Based on the results of this detailed study, a modified linear shrinkage test procedure was devised in which each of the variables investigated was controlled to obtain maximum linear shrinkage for each soil. It was hoped that the maximum linear shrinkage values obtained by this modified procedure would be more closely related to the shrink-swell potential results than the linear shrinkage test results obtained by the test procedure used in the original study (*1*). Eleven of the twelve soils used in the original study were tested by the modified procedure discussed in the following paragraphs.

A 75-gram sample of air-dry soil passing the No. 40 sieve was mixed 15 minutes in a milkshake machine with enough distilled water for the mixer to easily stir the mixture. (Some of the sandy soils were mixed with 1-Normal HCl solution to prevent separation of the sand and clay particles). The mixture was poured into Teflon molds, 20.0 cm. long by 2.54 cm. diameter (semicircular cross section) that had been previously lubricated with 0.3 gram of petroleum jelly. Drying was done in an oven at 70° C. ± 5° C.

The test results were disappointing. The linear shrinkage for each soil was more than in the original study (*1*), as shown in figure 14. However, the correlation of these maximum linear shrinkage values with shrink-swell potential was not quite as good as was obtained from specimens molded at the liquid limit. No further efforts were made to improve the correlation. The correlation results are reported here primarily to save the time and effort of other researchers who might consider trying the same approach toward improving the correlation. Also, the qualitative and quantitative measure of variables affecting linear shrinkage test results may prove useful to other researchers investigating the linear shrinkage test.

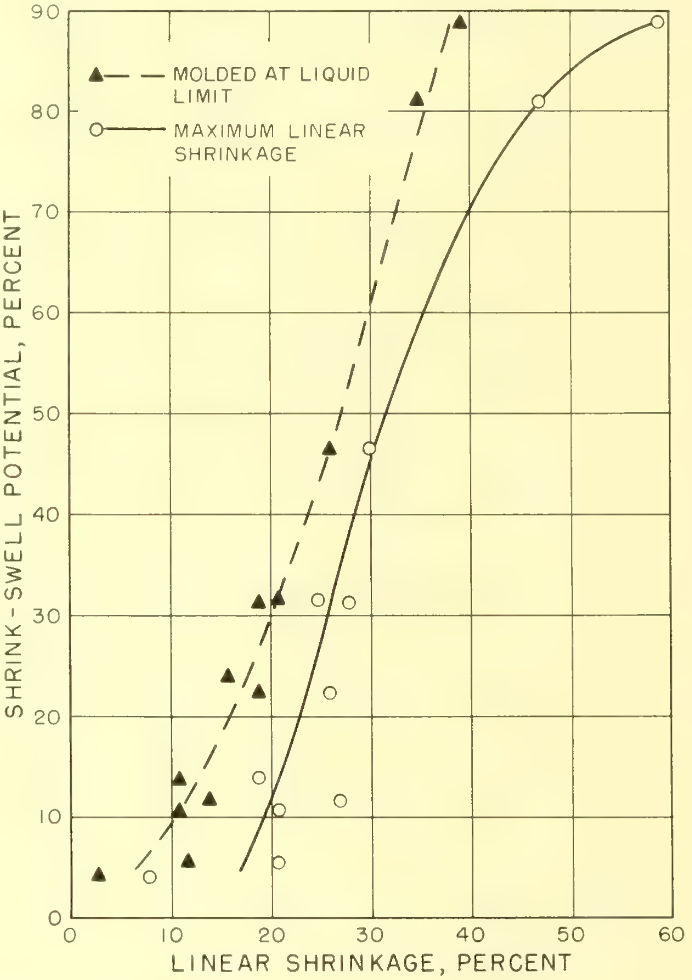

Figure 14.—Relation of linear shrinkage results obtained from two test procedures to shrink-swell potential.

Suggested Test Method for Determining the Linear Shrinkage of Soils

Definition

The linear shrinkage of a soil is the change in length of a bar of soil determined in accordance with the procedure outlined in the following paragraphs.

Apparatus

Apparatus needed for the test is:

- Linear shrinkage mold—a Teflon mold 20 cm. long and having a semicircular cross section 2.54 cm. in diameter.
- Commercial petroleum jelly.
- Distilled water.
- Evaporating dish, about 4½ inches in diameter.
- Balance, 500-gram capacity, sensitive to 0.1 gram.
- Spatula, having a blade about 3 inches long and about three-fourths inch wide.
- Drying oven, 70° C. ± 5° C.
- Scale, length 30 cm. graduated to one-half mm.

Sample

A sample of air-dry soil weighing about 150 grams shall be taken from the thoroughly mixed part of the material passing the No. 40 (420-micron) sieve.

Procedure

The soil sample shall be placed in the evaporating dish and thoroughly mixed with 45 to 60 cc. of distilled water by alternately and repeatedly stirring, kneading, and chopping with a spatula. Additions of water shall be made in increments of 3 to 8 cc. until the soil is at or slightly above its liquid limit (see AASHO T 89–60). Each increment of water shall be thoroughly mixed with the soil, as previously described, before another increment of water is added.

The mixture shall be placed in a linear shrinkage mold that has been previously lubicated with 0.30 gram petroleum jelly. After firmly pressing the mixture into the mold with the spatula, excess material shall be removed by trimming the mold with the straight edge of the spatula.

The mold containing the mixture shall be placed in an oven at 70° C. ± 5° C. for 16 hours or until constant weight has been obtained.

Note 1: When the oven must be set at 110° C. for other soil tests, linear shrinkage specimens may be dried at this temperature. However, the higher temperature will cause more cracking of the specimen and slightly lower test values.

The soil specimen shall be removed from the oven, allowed to cool, and then the length of both its top and bottom measured.

Note 2: For broken specimens, the lengths of the individual pieces should be marked and accumulated on a strip of paper; the total length can be determined directly by measuring the end points on the strip.

Calculation

The linear shrinkage of the specimen shall be expressed as follows:
Linear shrinkage (in percent) =

$$\left[\frac{\text{Mold length} - (\text{top} + \text{bottom length of dried specimen})/2}{\text{mold length}}\right] \cdot 100$$

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(*1*) *Shrink-Swell Potential of Soils,* by George W. Ring III, PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, vol. 33, No. 6, Feb. 1966, pp. 97–105.

(*2*) *Determining the Shrinkage Factors of Soils,* AASHO Designation: T 92–60, Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II. Methods of Sampling and Testing, The American Association of State Highway Officials, 1961, pp. 295–298.

(*3*) *Method of Determining the Shrinkage Factors of Soils,* THD–68, revised Item 5, Linear Shrinkage (Bar Method), by Texas Highway Department, Soil Testing Procedures, Materials and Test Division, 1953.

(*4*) *Developing a Procedure for Rapid Field Soil Testing and Classification,* by G. Hondros, Commonwealth Engineer, vol. 45, No. 4, Nov. 1, 1957, pp. 50–54.

Errata

Data shown in two figures in the article, *Severance Case Studies—Bridging the Gap Between Findings and Their Application,* by Floyd I. Thiel, which appeared in the April 1967 issue of PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, vol. 34, No. 7, were in error. Corrected figures 4 and 8 are reprinted here. Please make the substitution for figure 4 on page 147, and for figure 8 on page 149, of the April magazine.

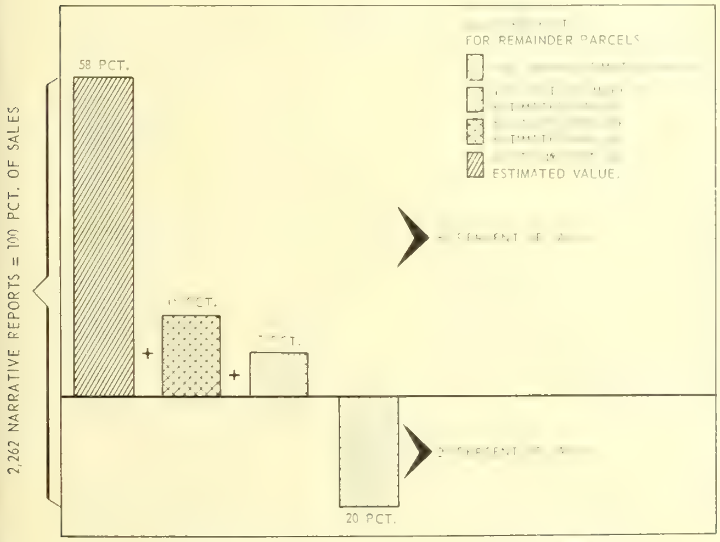

Figure 4.—Remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

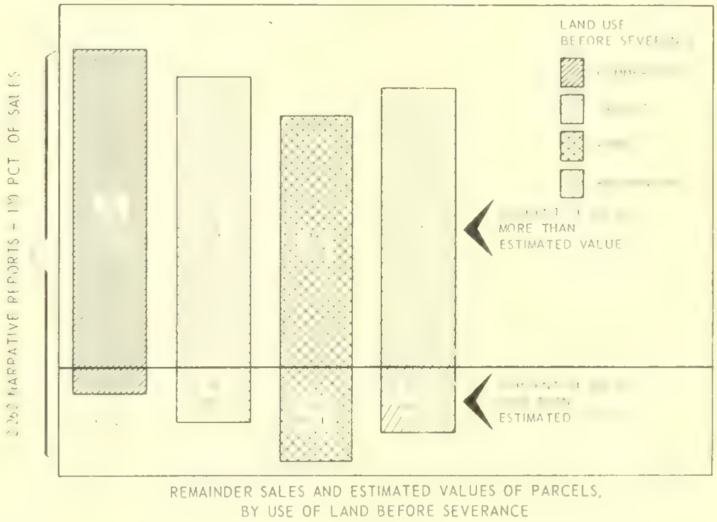

Figure 8.—Effect of land use before severance on remainder sales and estimated values, based on 2,262 narrative reports on severance studies.

Design Use of the Capacity Manual

BY THE OFFICE OF ENGINEERING AND OPERATIONS BUREAU OF PUBLIC ROADS

Reported by [1] DONALD W. LOUTZENHEISER, Chief, Highway Standards and Design Division, WILLIAM P. WALKER, Chief, Geometric Standards Branch, and DONALD B. LEWIS, Highway Engineer

The authors of the report presented here believe that the highway designer should make use of the Highway Capacity Manual because it makes possible a greater knowledge and understanding of all factors related to highway types, speeds, volumes, and operations than any other source. Use of the Manual is necessary for certain analyses, particularly for ramps and intersections at grade, but it provides broad ranges of values only, and does not furnish the designer with a specific value for use in actual design. However, the essential working data for most design analyses are available in condensed, usable form in the AASHO Policy on Geometric Highway Design (Blue Book). The Blue Book also gives national policy guidance on all main factors, except the new intersection data. For design use, the authors suggest the use of a peak hour factor of 0.85 for most cities, and a load factor of 0.3. The factors provide the missing guidance for intersection design. The authors advise that the AASHO Blue Book be used as the source for policy guidance and as a major tool for carrying out capacity analysis within the areas covered and that the Manual be used to supply supplemental information for design analyses.

Introduction

WHEN THE SWEDISH astronomer, Anders Celsius, presented the Centigrade scale for temperature measurement to the Swedish Academy of Science some two and a quarter centuries ago, he gave to the scientific world a device marked by simplicity, utility, and lasting endurance. Celsius arrived at his centigrade scale by dividing the temperature range for the liquid state of water under standard conditions into 100 equal parts. But, Celsius did not concern himself with the biologic effects that water at different temperatures would have on the human body, nor did he discover, deduce, or decree that water at 60° C. would destroy human tissue, or that a water temperature of 20° to 30° was most comfortable for body immersion, or that a human is not likely to survive longer than 2 hours if immersed to his chin in water at 5° above freezing. Neither is he credited with any discovery of the effect that variations from the standard condition, such as admixtures or changes in atmospheric pressure, might have on the boiling point or freezing point of water. These findings were left for scientists in other fields of interest.

In like manner, the Highway Capacity Committee of the Highway Research Board has given the highway engineering profession a valuable instrument, the *Highway Capacity Manual, 1965* (hereafter referred to as *Manual*), which gives a scale of measurement for highway operating conditions and deals with the effects of variations from standard conditions. The *Manual* scale used to measure the operating conditions that exist with different traffic loads is as simple as the Celsius centigrade scale. The range in possible highway operating conditions is divided into five, unequal quantitative parts and the parts are ranked from superior, at negligible traffic volume, to intolerable, at full capacity load. These quantitative parts are called levels of service and are lettered A through E, from good to bad. The Highway Capacity Committee attempted to give an adjective-type description of each level of service and avoided any recommendation as to how good the service provided by the highways should be. This was done in recognition of the fact that the level of service is governed as much by economics as by the desires of highway users. Therefore, the *Manual* is a valuable research report but it is not a national policy or guide that can be used directly for design.

The *Manual* must be supplemented by design guidelines before it can be applied in practice. The *AASHO Policy on Geometric Design of Rural Highways, 1965* (referred to as Blue Book) provides most of the needed guidance. The authors of the report presented in this article advise that this Blue Book is the essential handbook for the highway designer and the *Manual* is a necessary academic supplement. However, as discussed later, the Blue Book does not contain all the needed data for design of ramps and intersections at grade. In the interest of simplicity, brevity, and practicality, the Blue Book has compromised with precision in many places where the AASHO Committee determined that the accuracy of the result would be within the limits of accuracy of the estimated traffic data upon which the design would be based.

Manual factors and procedures suggest accuracy of details in capacity analyses that may be beyond the practical need for design; nothing is gained in stressing such details that are inconsistent with the reliability of highway section data or the practical use of analyses conclusions. Design data from the Blue Book may be applied to computational procedures in the *Manual* to determine geometric dimensions that satisfy operational requirements for a given volume of traffic, or to determine the level of service that an existing highway will afford any given volume of traffic. Examination of the material from each book will show the reasonableness of the design data and procedures.

Explanation of Terms

The *Manual* has been written with some terms that are new or somewhat different than those designers have been using. The single word *capacity* refers to the upper limit or possible capacity, as previously known. It also is defined as the maximum number of vehicles that can reasonably be expected to pass over a given section of a lane or a roadway in one direction, or in both directions for a 2-lane highway, during a given time period under prevailing roadway and traffic conditions.

Level of service denotes any one of an infinite number of different combinations of operating conditions that may occur on a given lane or roadway when it accommodates different traffic volumes. Each service level is a measure of the effects of operating factors such as speed, traveltime, freedom to maneuver, safety, operating costs, and so on. For uninterrupted flow, that is, where traffic movement is not interrupted by stop signs or signals, the levels of service are defined by operating speed related to the volume-to-capacity ratio. Application of this volume-to-capacity ratio to the 2,000 vehicles per hour (v.p.h.) lane capacity for multilane highways gives service volume limits of average lane volumes:

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[1] Presented at the 46th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1967.

Operating speed and service volume for each level of service for multilane highways other than freeways

Level of service	*Operating speed, m.p.h.*	*Average lane volume, number of cars*
A	70–60	0– 600
B	60–55	600–1,000
C	55–45	1,000–1,500
D	45–35	1,500–1,800
E	35–30	1,800–2,000

As generally used, a *service volume* is the maximum number of vehicles that can pass over a given section of lane or roadway, one direction on multilane highways or both directions on 2-lane highways, during a specified time period while operating conditions are maintained at a specified level of service. *Service volume,* generally an hourly volume, is the limiting volume for that level of service, and corresponds to the *design capacity* for the determined level of service.

Average highway speed is the weighted average of the design speeds within a highway section. *Operating speed* is the highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions, without exceeding the safe speed used for design. In the Blue Book, the data are shown in terms of the relation of design speeds to average running speeds and in the *Manual*, the relations are in terms of average highway speeds and operating speeds; however, the overall data are essentially the same. Within the limits of practical design, operating speed, as used in the *Manual,* is the same as average running speed plus 4 to 8 m.p.h., as used in the Blue Book.

Peak hour factor (PHF) is the ratio of the volume of traffic occurring during the peak hour to the maximum rate of flow during a part of the peak hour, usually 5 minutes for operation on freeways and 15 minutes on intersections. In the *Manual* this PHF is applicable to urban freeways at service levels C and D and to intersections, but not to rural freeways. A low PHF indicates a sharp peaking characteristic within the hour. The upper or theoretical unity value of PHF is never attained in practice. Common PHF values are about 0.8 to 0.9. *Load factor* at signal controlled intersections is the ratio of the total number of green signal intervals that are fully utilized by traffic approaching the intersection from one direction during the peak hour to the total number of green intervals within the hour. Common load factor values within the design range are 0.2 to 0.6.

Capacity values in the *Manual* are established in terms of maximum numbers of passenger cars per hour under ideal conditions. To derive the values for highway capacity where conditions are not ideal, adjustment factors are applied to the upper limit in a series of downward corrections. Adjustment factors include lane width, lateral clearance, trucks and grades, buses and grades, alinement, peak hour factor, traffic interruptions, passing sight distance, and load factor. However, of these, some factors are not applicable to all classes of highways. In the following discussion, only the maximum passenger car per hour values are cited. The fact that the adjustment factors need to be applied is not repeated because the application of adjustment factors always is the basic procedure. The same is true of the design capacity values cited from the Blue Book.

Uninterrupted Flow

Procedures are given in the *Manual* for the calculation of uninterrupted flow capacities on different types of highways: (1) Freeways and other expressways; (2) multilane highways having no access control; and (3) 2-lane highways.

Freeways and other expressways

Freeways and other expressways are controlled-access highways and have 4 or more lanes. Maximum service volumes—the design capacities—given in the *Manual* for the three levels of service within the framework of normal design, are listed in table 1. The average service volume per lane varies substantially in relation to the number of lanes. Depending on the PHF, the volumes of vehicles during any 1 hour may be considerably less than the maximum volumes quoted in table 1 for the levels of service. The *Manual* recommends that, for levels C and D, the maximum service volumes should not be exceeded during the peak 5 or 6 minutes of the hour. Also, the *Manual* notes a variation in service volume in relation to the alinement characteristics of the highway, expressed in terms of the average highway speed. The Blue Book covers the same variation in service volume by relating the average running speed to the design speed; this is a less precise relationship but one more convenient to use in design, where the average highway speed cannot be readily determined.

Because of the three major variables, number of lanes, peak hour factor, and average highway speed, service volumes may vary over a significant range up to the maximum values given in table 1. It is fortunate that in most of today's freeway design work, these three variables fluctuate within a rather narrow range because the designer probably would not have the necessary information on peak hour factor and average highway speed to make the precise calculations described in the *Manual.* Rounded values for design capacities of freeway lanes, which will fit most situations without appreciable error, are given in the Blue Book.

The average lane value of 1,000 vehicles per hour is applicable to rural freeways in level and rolling terrain according to the Blue Book. This value is within the range of level of service B as described in the *Manual* for 4-, 6-, and 8-lane freeways with 70 m.p.h. design characteristics. On such freeways the service volume will permit an operating speed of 55–60 m.p.h., corresponding to an average running speed of 45–50 m.p.h. The average lane value of 1,200 vehicles per lane per hour as given in the Blue Book is applicable for freeways approaching urban areas and those in mountainous terrain. With this volume of traffic most freeways, regardless of number of lanes, will provide level of service C, unless the design speed is below 60 m.p.h. and then level D will be achieved. The average lane value of 1,500 vehicles per lane per hour in the Blue Book is applicable for freeways in urban areas. This volume will result in level of service D in most instances. If the design speed is below 50 m.p.h. and the alinement is continuously winding, operation would fall slightly below level D. If the design speed were 70 m.p.h., an 8-lane freeway would provide a service slightly better than level D, but a 4- or 6-lane freeway would provide only level D. Some years ago it was concluded that freeways designed on the basis of these hourly values of traffic per lane would yield satisfactory service at reasonable cost for construction; experience has confirmed this conclusion.

Table 1.—Maximum service volumes for 3 levels of service within the normal design range

Level of service	Average maximum service volume, 1-way		
	2-lane	3-lane	4-lane
	v.p.h.	*v.p.h.*	*v.p.h.*
B	2,000	3,500	5,000
C [1]	3,000 (2,300)	4,800 (3,700)	6,600 (5,100)
D [1]	3,600 (2,800)	5,400 (4,150)	7,200 (5,600)

[1] Numbers in parentheses show the lower limit of service volumes under ideal conditions as governed by a peak hour factor of 0.77.

Multilane Highways Without Access Control

The maximum service volumes given in the *Manual* for multilane highways without access control are 1,000 vehicles per lane per hour for service level B, 1,500 vehicles for C, and 1,800 vehicles for D. These maximum service volumes are the same as the average volumes per lane given in the *Manual* for 4-lane freeways for the same levels of service. For levels C and D the operating speed is said to be about 5 m.p.h. lower than that for freeways having the same service volumes. For freeway operation the *Manual* data for levels C and D give different service volumes for different numbers of lanes and for fluctuations within the hour, that is, the peak hour factor. However, for multilane highways without control of access, the service volumes per lane are the same regardless of the number of lanes and there are no peak hour factor adjustments.

In the Blue Book the design capacities for rural multilane highways and rural freeways are the same except that rural multilane highway capacities are reduced by the interference from cross traffic and roadsides. The maximum design capacity of 1,000 to 1,200 vehicles per lane on rural multilane highways where there is little or no interference may be within level of service B or C, the same as freeways depending on the average running speed. The maximum design capacities of 700 to 900 vehicles per lane with moderate interference and 500 to 700 vehicles with considerable interference reflect the effect of unsignalized intersections at grade and roadside interference.

2-lane highways

The 2-lane highways have a restriction not existing on multilane highways, which is the result of the need for traffic occupying the opposing lane when passing. Therefore the percent of passing sight distance available along the route must be determined. Service volumes or design capacities are the total for both directions regardless of the distribution of traffic. The maximum service volume given in the *Manual* for level of service B is 480 to 900 vehicles in both lanes; for level C, 1,080 to 1,400 vehicles; and for level D, 1,600 to 1,700 vehicles. The range of service volumes reflects the effect of available passing sight distance, the higher volumes each time being applicable where sight distance is not restricted.

Maximum service volumes

The maximum service volumes for the levels of service given in the *Manual* correspond approximately with the maximum design capacities for the average running speeds given in the Blue Book. The maximum design capacity for an average running speed of 45 to 50 m.p.h. is 480 to 900 vehicles; for 40 to 45 m.p.h., 680 to 1,150 vehicles; and for 35 to 40 m.p.h., 1,110 to 1,500 vehicles. The indicated range of design capacities reflects the effect of available passing sight distance that is always applicable where there are no sight distance restrictions. The maximum design capacity for the 45 to 50 m.p.h. running speed is the same as the maximum service volumes of level B given in the *Manual*. This average running speed and corresponding design capacity is recommended for application in the design of most main rural 2-lane, 2-way highways in level and in rolling terrain. The maximum design capacity for the 40 to 45 m.p.h. average running speed corresponds approximately to level C. This average running speed is recommended for application in the design of 2-lane highways approaching urban areas and, wherever feasible, for 2-lane highways in mountainous terrain. The maximum design capacity for the 35 to 40 m.p.h. average running speed corresponds approximately to level D. This average running speed is recommended for application in the design of 2-lane rural highways in mountainous terrain where design for higher level of service is not feasible. The reduction of these design capacities by the several adjustment factors that are applicable to the service volumes in the *Manual* has been largely accomplished in a series of tables (tables II–8 through II–10) of the Blue Book.

Intersections at Grade

The method used in the 1965 *Manual* for determining the service volumes, as now termed, for approaches to intersections is essentially the same as used in the 1950 *Manual*. The source data for the most recent *Manual*, however, have been based upon much more research. New adjustments have been added, particularly the peak hour factor—fluctuation within the hour—, the load factor—loaded cycles within the hour—, and metropolitan area size. Level of service is defined in terms of selected load factors, that is, the larger the number of loaded cycles within the design hour, the poorer the level of service.

The Blue Book does not furnish capacity design values for intersections, but refers the reader to the *Manual*. The procedures and factors shown in the *Manual* should be used directly for design. But if design is to be on a uniform basis, the user of the *Manual* must have some guidance in selecting the peak hour factor and the load factor values. Studies should be made to determine local values for these factors for the type of intersection being designed. Where such data are not available general use of a peak hour factor in the range of 0.80 to 0.90 and a load factor of 0.3 is recommended. In cities of 1 million or more the PHF should be about 0.90 and for cities of less than 100,000 about 0.80. For cities within the range of 100,000 to 1 milllon a rounded value of 0.85 is suggested but the user is at liberty to make a more exact interpolation if he chooses. The load factor of 0.3 is that of level of service C, which should be a justifiable realm of design. With these guides, intersection capacity analysis follows the same general procedures and details as have been used in the past. Many designers found that the intersection capacity charts published in *PUBLIC ROADS, A Journal of Highway Research*, February 1951, made the analysis simple and assuredly complete. A similar set of revised charts is expected to be published in the August 1967 issue of this magazine.

Weaving Areas

The level of service afforded by a weaving section is a function of the number of lanes and of the length of the weaving area. The relation of length to level of service and service volume is shown in figure 7.4 of the *Manual*. The needed number of lanes is determined by an equation. The curves on the graph that illustrate the relationship of needed length of weaving section to the volume of weaving traffic are numbered for correlation to the level of service for different highway conditions. This correlation is shown in table 7.3 of the *Manual*. In determining the number of lanes needed for weaving sections, an equation is used in which the smaller of two weaving volumes is multiplied by a K factor. In the *Manual* the K factors are shown on the weaving chart. In the equation the service volume value (SV) is the service volume per lane used in determining the number of lanes for the approach roadway.

Weaving values obtained from two charts in the Blue Book (figs. IX–16 and IX–17) agree with those from the *Manual* for the two levels of service covered by the Blue Book. But, the information is presented somewhat differently. The 40 m.p.h. curve in figure IX–16 corresponds to curve III in the *Manual*; the 30 m.p.h. curve in figure IX–17 corresponds to curve IV in the *Manual*. In the Blue Book, K factors for use in the equation for determining the number of lanes needed for weaving sections are obtained from supplemental charts.

The C value used in the formula in the Blue Book for determining the width of the weaving section is the design capacity per lane of the approach roadway. C values of 1,000, 1,200, and 1,500 with appropriate adjustments are given in the Blue Book for rural, suburban, and urban respectively, as described in connection with freeways. The C value or design capacity per lane of the approach roadway, may also be used as service volume (SV) if the *Manual* equation is used and the results obtained by the two methods are the same.

Ramps

The capacity of either terminal or the capacity of the cross section of the ramp can control ramp capacities. Usually, ramp capacity is controlled by the terminal area. Ramp terminals fall into three broad categories, at-grade intersections, merging areas, and diverging areas. At-grade intersections may be solved by the procedures already discussed. The *Manual* gives two procedures for determining the capacity of merging and diverging areas, one for determining levels of service A, B, and C, the other for levels D and E. Usually design should provide for level C or better, therefore use of the first procedure for design is appropriate. The maximum service volumes for lane 1, including ramp traffic and through traffic, used for level of service B is 1,200 vehicles merging and 1,300 vehicles diverging; and for level of service C is 1,700 vehicles merging and 1,800 vehicles diverging. The volume of through traffic in lane 1 depends on a number of variables and may be determined by the series of nomographs (figs. 8.3 to 8.19) or equations included in the *Manual*.

The Blue Book recommends use of the equations and nomographs in the *Manual*. It specifies the maximum volumes in lane 1 for use in design: 1,200 vehicles merging and 1,300 vehicles diverging for rural highways and 1,500 vehicles merging and 1,600 vehicles diverging for urban highways. These maximum volumes correspond to the level of service B for rural areas and are within level C for urban areas. The volumes as given in the *Manual* and Blue Book differ slightly in that the volumes given in the *Manual* include 5 percent trucks on an assumed level grade and the volumes given in the Blue Book are in terms of equivalent passenger cars.

NEW PUBLICATIONS

Highway Statistics, 1965

The 21st issue of the annual compilation of statistical and analytical tabular matter pertaining to Federal-aid for highways, *Highway Statistics, 1965*, has recently been issued by the Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation. This publication may be ordered from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, for $1.00 a copy, prepaid.

Highway Statistics, 1965, a 182-page publication, presents information, primarily in tabular form, on motor fuel, motor vehicles, driver licensing, highway-user taxation, State and local highway financing, road and street mileage, and Federal-aid for highways.

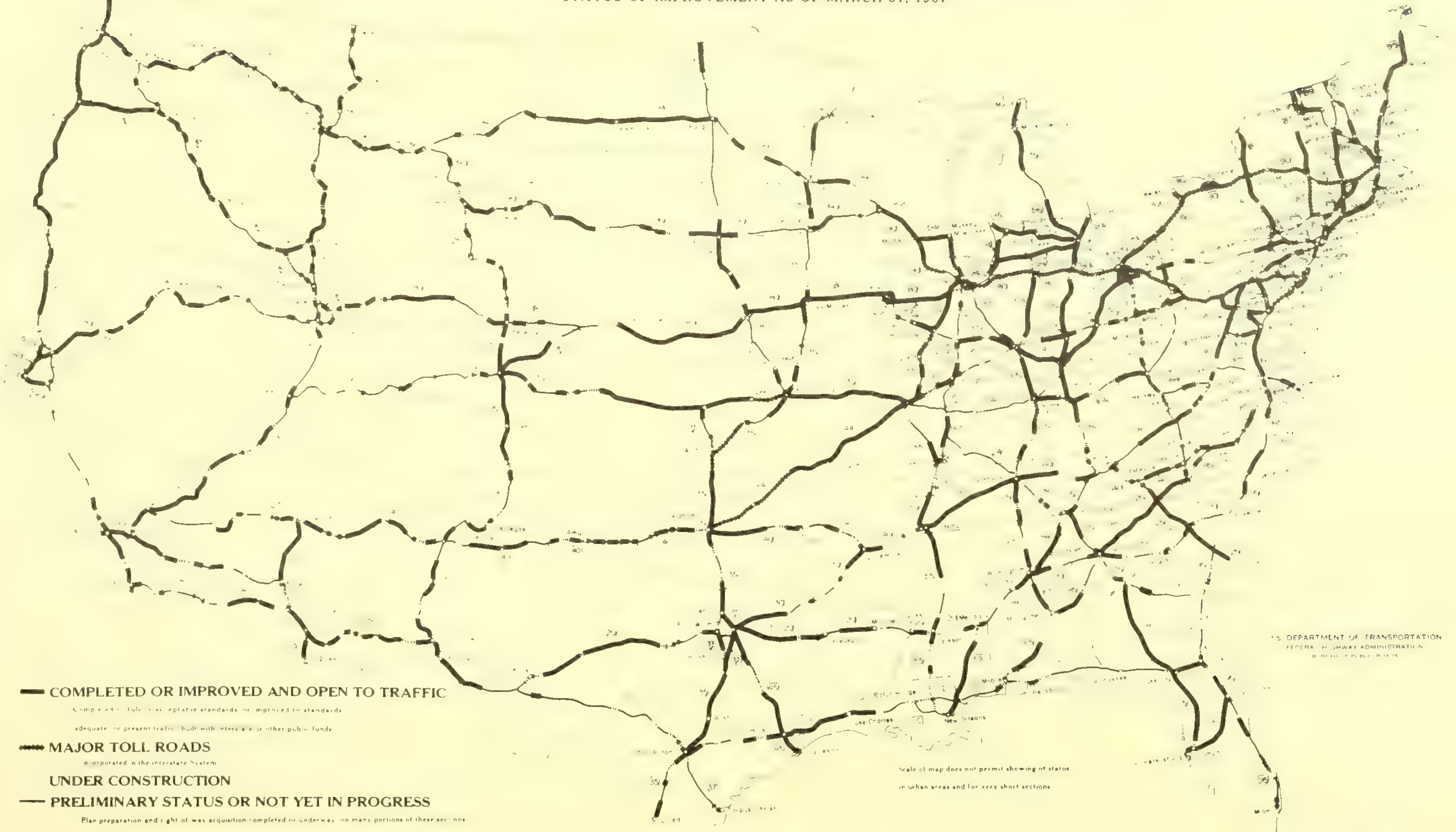
Highway Statistics, Summary to 1965

Highway Statistics, Summary to 1965, an historical summary of information on highways, their use, and financing has recently been published by the Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation, and is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402. The 178-page publication is a comprehensive statistical review of highway development in the United States through 1965, and it includes all information previously presented in the Highway Statistics summaries for 1945 and 1955, as well as information published in the annual issues of *Highway Statistics* through 1964.

Highway Statistics, Summary to 1965, contains statistical and analytical tables on four major subject areas. The data on motor fuel include analysis of motor-fuel consumption, tax rates, and tax receipts. The section on motor vehicles includes tables on motor-vehicle registration and operator's licenses, their fee schedules, and the revenues received therefrom and from motor-carrier taxes; this section also includes travel data. The highway finance section covers the disposition of highway user imposts, receipts, and expenditures for highways and highway debt. Because of the interest in the subject, data for toll facilities are presented separately. For the first time, the summary includes the local highway finance and related data; these data were previously given only in the annual bulletins. The mileage section includes reports on road and street mileage existing and the mileage constructed each year, classified by system and type. The section on Federal-aid includes tables on Federal excise taxes and on Federal-aid funds, construction, and system mileage.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF MARCH 31, 1967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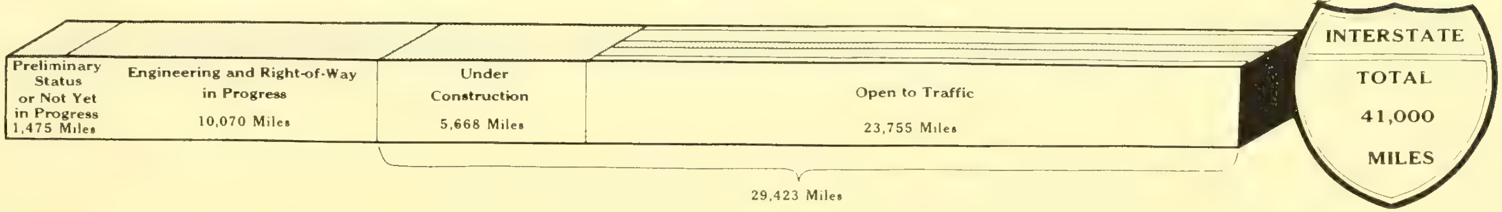
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS
Baton Rouge
Lake Charles
New Orleans
Scale of map does not permit showing of status
in urban areas and for very short sections
COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards or improved to standards
adequate for present traffic, built with Interstate or other public funds
MAJOR TOLL ROADS
Incorporated in the Interstate System
UNDER CONSTRUCTION
PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right of way acquisition completed or underway on many portions of these sections



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REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:
Supplementary Report, House Document No. 124 (1965). $1.00.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42 (1965). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61, $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

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Highway Bond Financing . . . An Analysis, 1950–62. 35 cents.

Highway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

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PUBLICATIONS—Continued

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (1966). $1.00.

Highway Statistics (published annually since 1945):
1966, $1.00.
(Other years out of print.)

Highway Statistics, Summary to 1966.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways to Beauty (1966). 20 cents.

Highways and Economic and Social Changes (1964). $1.25.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). Out of print. Appendix, 70 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

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Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

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Standard Plans for Highway Bridges (1962):
Vol. III—Timber Bridges. $1.00.
Vol. IV—Typical Continuous Bridges. $1.00.
Vol. V—Typical Pedestrian Bridges. $1.75.

Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

Traffic Assignment and Distribution for Small Urban Areas (1965). $1.00.

Traffic Assignment Manual (1964). $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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AUGUST 1967

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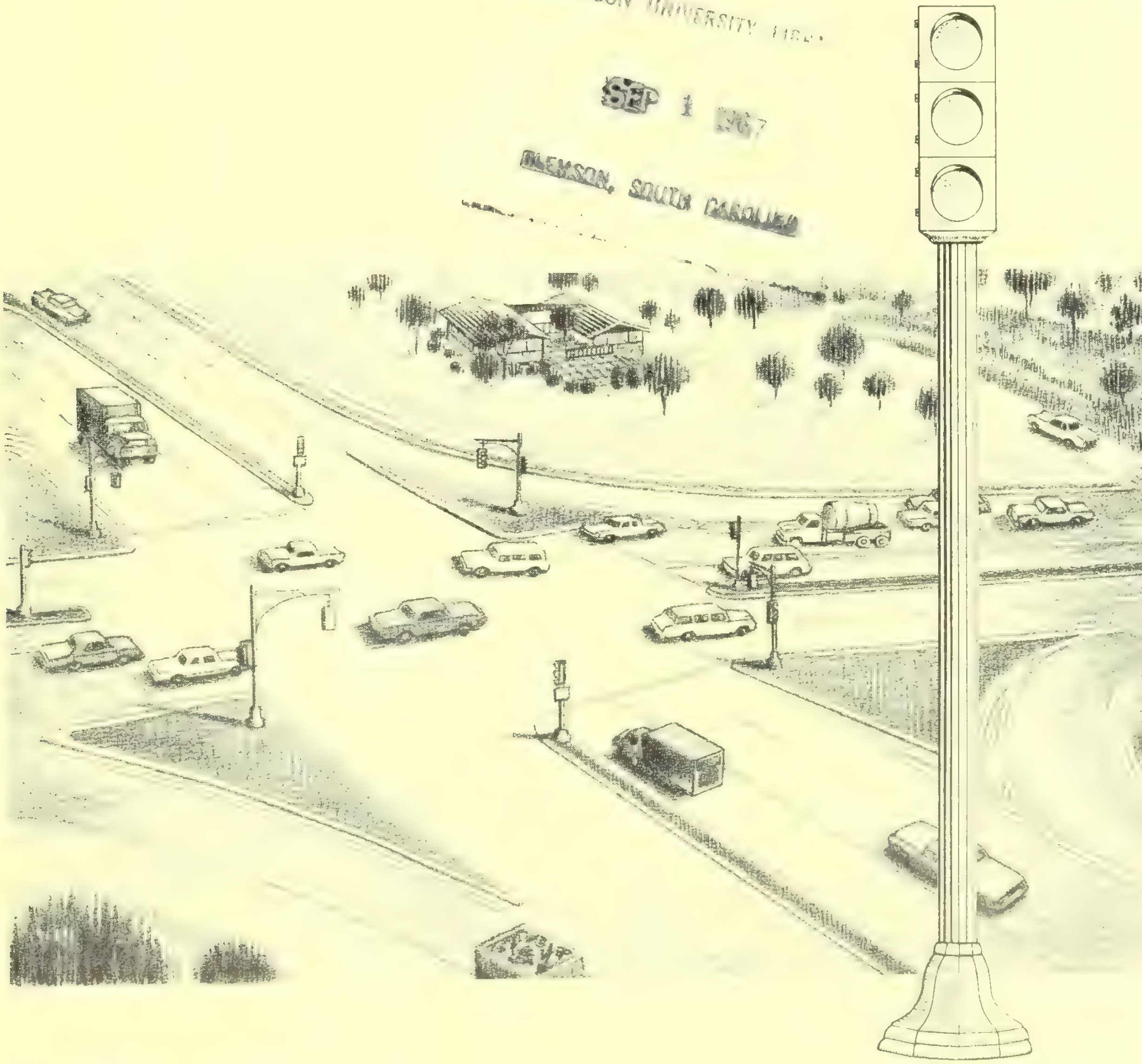

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SPECIAL ISSUE

Design capacity charts for signalized intersections

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Capacity Analysis Techniques for Design of Signalized Intersections

Installment No. 1

Sponsored by the
OFFICE OF ENGINEERING AND OPERATIONS
BUREAU OF PUBLIC ROADS

by [1] JACK E. LEISCH, Vice President and Chief Highway Engineer, DeLeuw, Cather & Co. of Canada Ltd.

INTRODUCTION

After publication of the 1950 Highway Capacity Manual, the author presented a procedure for the graphic solution of the capacity of signalized intersections to simplify the work required by the computational procedures in the Manual. Publication of the 1965 Highway Capacity Manual has provided a revised and comprehensive basis for computations of signalized intersections. Many users of the graphic procedure had expressed a desire for revised charts that reflected the revisions in the new Manual.

In this article, which will be presented in two installments, the author has again filled the need for a graphic procedure incorporating current knowledge. In addition to updating the original charts, new charts have been prepared to cover capacity procedures for which calculations previously required extensive application of judgment. Full discussion of the principles and procedures in the application of the charts in addition to sample problems have been included. The information presented provides a graphic procedure for the capacity analysis of most signalized street and highway intersections.

The current set of charts comprises 20 nomographs. Eighteen of the nomographs are presented in this first installment together with the appropriate application procedures and sample problems. The other two nomographs and the remainder of the article will be included in the second installment to be published in the October issue, Vol. 34, No. 10, of PUBLIC ROADS.

HIGHWAY CAPACITY has become recognized as an essential discipline of highway planning and design. Its importance became apparent with the publication of the 1950 *Highway Capacity Manual* (*1*),[2] which has been superseded by a 1965 *Highway Capacity Manual* (*2*), hereafter referred to as the *Manual*. This new publication was based on considerable research and operational experience; it is a sophisticated and highly useful tool for the traffic and highway engineer in the planning, design, and operation of highways and highway systems.

Capacities of streets and the many interchanges associated with freeways in urban areas are determined largely by the at-grade intersections and ramp terminals. To make less cumbersome the capacity analyses required for design of intersections, a set of nomographs based on the 1950 *Highway Capacity Manual* was prepared and published in a 1951 issue of *Public Roads, A Journal of Highway Research* (*3*). Because of the popularity of the nomographic procedure and its adoption by numerous agencies, a revised set of charts, based on the data of the 1965 *Manual* has been developed. These charts and explanations for their use are reproduced in this article.

Development of Capacity Charts

The nomographs printed here generally conform to the format of those originally charted (*3*) so that a person familiar with the previous charts can use the new charts without having to learn a different procedure. The nomographs have been devised to simplify and reduce the work that otherwise would be required by the long-hand, computational procedures set out in chapter 6 of the *Manual*.

The basic data for intersection capacities are shown in figures 1 and 2 for 1-way and 2-way streets, without parking and with parking, expressed in terms of volumes on one approach—vehicles per hour of green—observed for different conditions of loading, metropolitan area size, peak hour factor, and location within the metropolitan area. The results are representative of average conditions derived from actual observations of some 1,100 intersection approaches. The average conditions are indicative of 10-percent right turns, 10-percent left turns, 5-percent trucks and through buses, and no local bus stops for pickup and discharge of passengers in the vicinity of the intersection approach.

Attainable volume of discharge (service volume) per approach, shown in figures 1 and 2, for a given or desired degree of loading must be adjusted for specific conditions. The additional adjustment factors are applied as multipliers in accordance with the values and instructions described in the *Manual*, pages 138–143. These include adjustments for the G/C ratio (signal timing), turning movements, proportion of trucks and through buses, and

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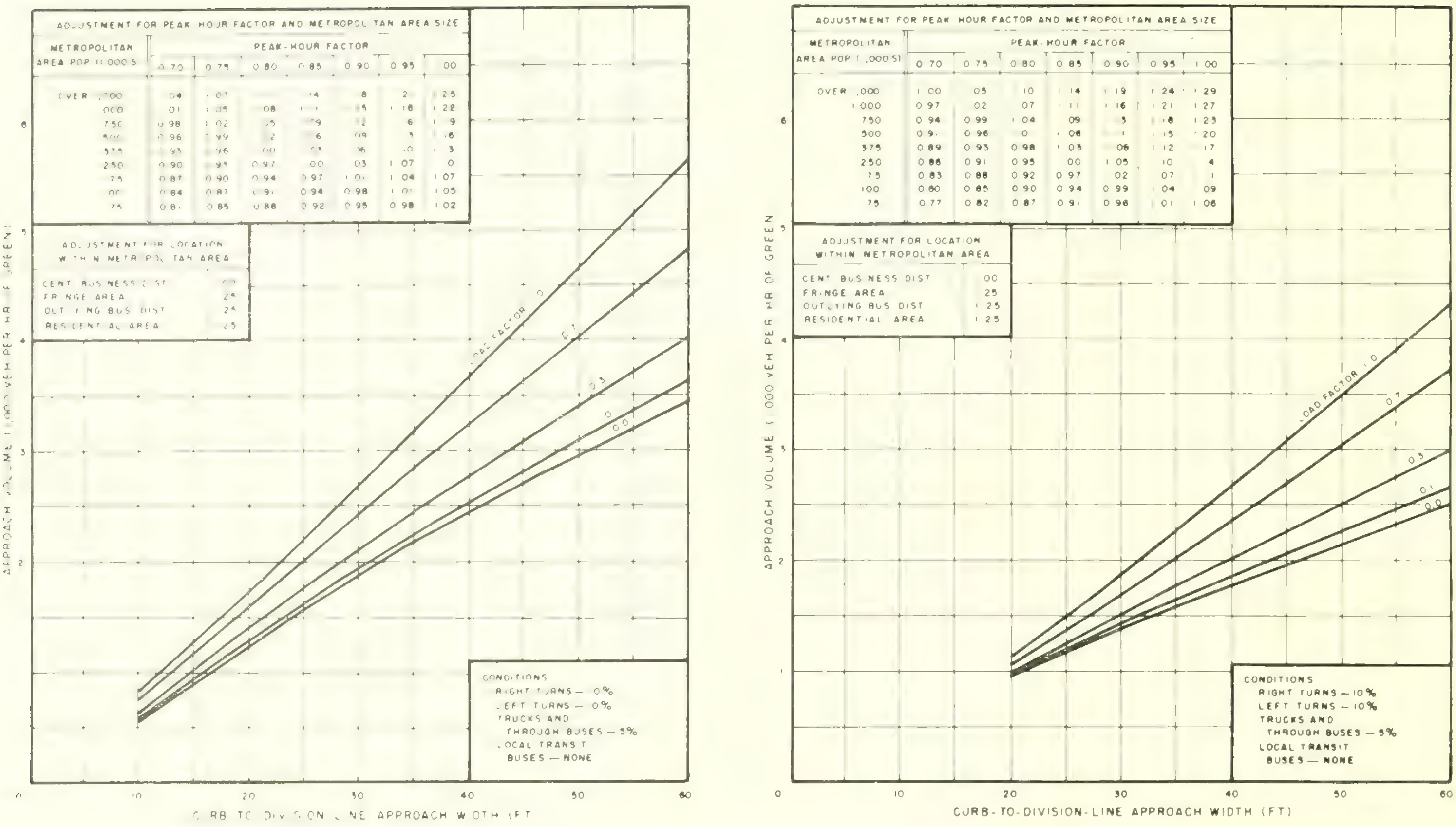
[2] References identified by italic numbers in parentheses are listed on p. 208.



A. Two-way street without parking
(Manual Fig. 6.8)

B. Two-way street with parking
(Manual Fig. 6.9)

Figure 1.—Urban intersection approach service volumes, basic data for 2-way streets.

bus stops. The charts presented here incorporate all of these adjustments, so that for any known condition, the intersection capacity can be obtained directly without reference to the *Manual* adjustment values. In constructing the charts, all of the adjustments have been precisely accounted for and no short cuts or approximations have been made, except for slight consolidation of values in the right- and left-turn adjustments in charts 3–15. Therefore, the results obtained by the use of these charts, for all practical purposes, are the same as those that would be obtained by use of the method in the *Manual*. Several refinements and additional (rational) procedures for conditions not covered in the *Manual* are also presented. These are included under *Special Conditions, Part 3,* and in charts 17 and 18 for separate turning lanes.

Capacity and Levels of Service

The *1950 Highway Capacity Manual* (*1*) related the ability of a highway to carry traffic to two levels—*practical capacity* and *possible capacity*. *Practical capacity* is the maximum number of vehicles that can be accommodated under prevailing roadway and traffic conditions without unreasonable delay or restriction to the driver's freedom to maneuver. *Possible capacity* represents the maximum number of vehicles that can be accommodated under prevailing roadway and traffic conditions, regardless of the effect of delaying drivers and restricting their freedom to maneuver.

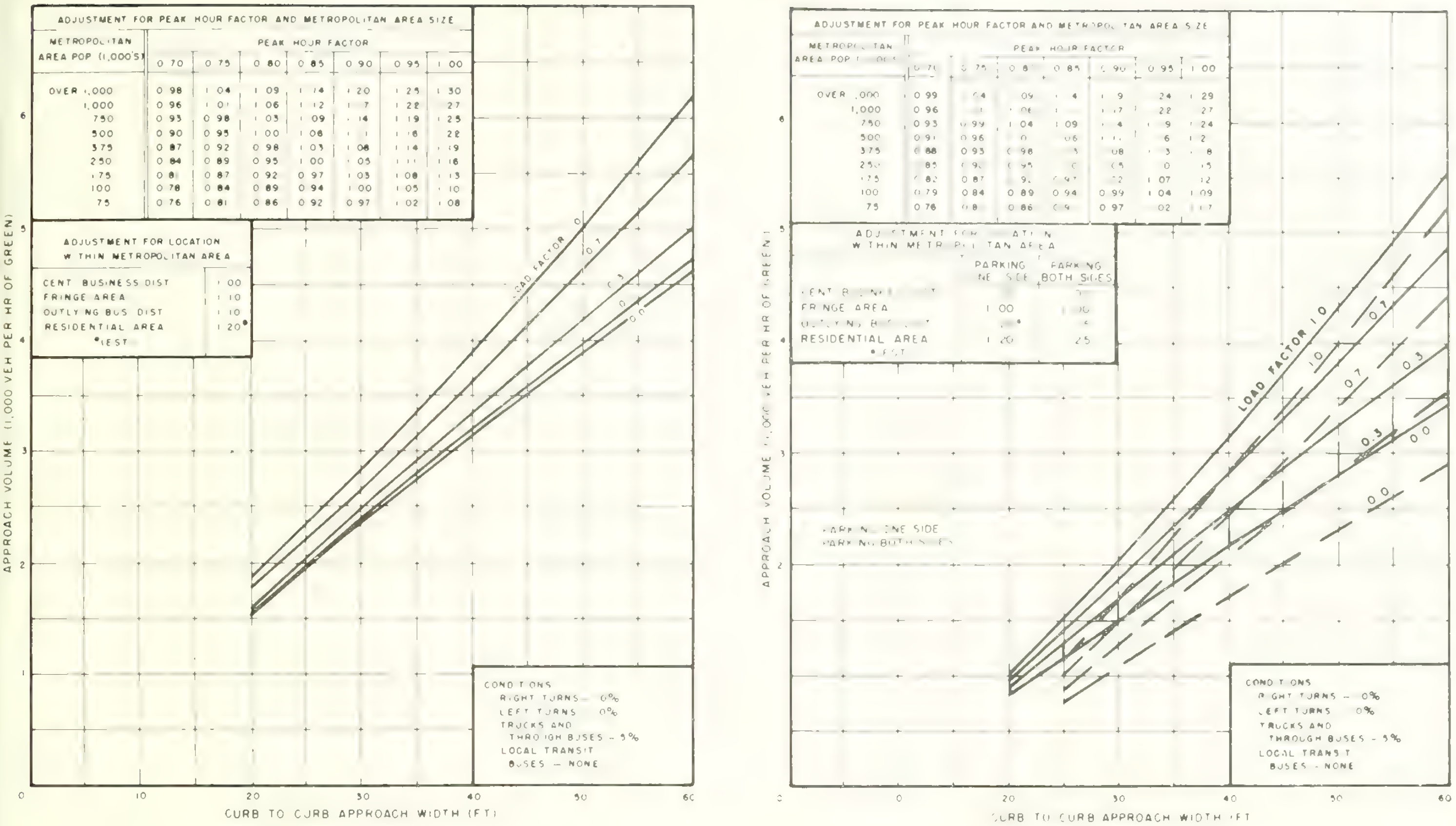
Design policies of the American Association of State Highway Officials (AASHO), published during the 1950's, accepted this concept and used the same terms for *capacity*. The AASHO practice, however, produced an additional term *design capacity* defined as the *practical capacity* or lesser value determined for use in designing the highway to accommodate the design volume. The original capacity charts (*3*) were predicated on *design capacity,* an adjusted value generally less, numerically, than *practical capacity,* whereas *possible capacity* was evaluated by application of appropriate conversion factors to *design capacity*.

The *Manual* (*2*) introduced the *level of service* concept, eliminated the terms *practical capacity* and *design capacity,* and substituted the single word *capacity* for what had been referred to as *possible capacity.* The six *levels of service* A through F designated in the *Manual* are a qualitative measure of operating conditions from excellent to intolerable, including *capacity. Level of service* constitutes the composite effect of speed and traveltime, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. An attainable hourly volume of traffic, or a maximum *service volume,* is designated in the *Manual* for each *level of service.* A *service volume* is the maximum number of vehicles that can be accommodated during a specified time period while operating conditions are maintained that correspond to the selected or specified *level of service.*

Table 1.—Levels of service as related to load factor for individual isolated intersection approaches [1]

Level of service	Traffic flow description	Load factor
A	Free flow	0.0
B	Stable flow	≤0.1
C	Stable flow	≤0.3
D	Approaching unstable flow	≤0.7
E	Unstable flow—capacity	≤1.0
F	Forced flow	----

[1] *Highway Capacity Manual,* 1965, p. 131.



A. One-way street without parking
(Manual Fig. 6.5)

B. One-way street with parking
(Manual Figs. 6.6 and 6.7)

Figure 2.—Urban intersection approach service volumes, basic data for 1-way streets.

The 1965 AASHO Policy on Geometric design of Rural Highways (*4*) refers to the *Manual* for basic values, but as in the previous policies, continues to use the terms *design capacity* and *possible capacity*. Although the terminology is different, the overall concepts in each publication are compatible. For example, the AASHO terminology *design capacity* is the same in essence as the *Manual* terminology *maximum service volume* for a selected *level of service*. Also, numerically, the AASHO terminology *possible capacity* is identical to the *Manual* terminology *capacity*. The relations presented in the charts, therefore, are equally applicable to the *Manual* and to the AASHO procedures.

The *load factors* ranging from 0.0 to 1.0, as shown on the curves of the basic data in figures 1 and 2, are indicative of *levels of service*. The *load factor* is a ratio of the number of green signal intervals that are fully utilized by traffic during the peak hour to the total number of green intervals for that approach during the same period. For intersection conditions, the *Manual* considers the load factor as an appropriate measure of the *levels of service*, since the loading is something the driver sees and interprets in terms of degree of congestion. The relation is shown in table 1.

Level of service B (*load factor* of not more than 0.1) is considered in the *Manual* to be suitable for design of intersections under typical rural conditions. *Level of service* C (*load factor* of not more than 0.3) normally is recommended for design of intersections in urban areas. *Level of service* E with operation at a load factor of 0.85 is taken to be representative of *possible capacity*. Although a factor of 1.0 sometimes may be approached, a lesser factor such as 0.85 generally indicates the maximum loadings that can be achieved repetitively and sustained over a period of 1 hour. Using these load factors, the relation between *design capacity* and *possible capacity*, for rural intersections and for different street types in urban areas, are summarized in table 2. The values shown are the ratios of attainable volumes per hour of green (average conditions) at 0.3 load factor (0.1 for rural conditions) to the attainable volumes at 0.85 load factor. Therefore, *design capacity* can be converted directly to *possible capacity* by multiplying *design capacity* by the appropriate factor (f) in table 2.

Table 2.—Factor *f* for conversion of design capacity to possible capacity

Street type and parking conditions	Factor f[1] when W_1, in feet, is								
	10	15	20	25	30	35	40	50	60
2-way:									
No parking	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
With parking			1.10	1.14	1.18	1.21	1.25	1.31	1.34
Rural[2]	1.28	1.28	1.28	1.30	1.32	1.35	1.38	1.41	1.44
1-way:									
No parking			1.15	1.13	1.12	1.12	1.13	1.15	1.17
Parking one side			1.10	1.13	1.16	1.18	1.20	1.25	1.30
Parking both sides				1.25	1.25	1.25	1.27	1.32	1.37

[1] Ratio of attainable volume at 0.85 load factor to that at 0.30 load factor, except for rural conditions
[2] Ratio of attainable volume at 0.85 load factor to that at 0.10 load factor

Although the charts are based on *design capacity* (*service level* C for urban conditions and *service level* B for rural conditions), conversion to any other *level of service* can be achieved by the use of factors (f) in table 3 for 2-way facilities and in table 4 for 1-way facilities. Thus, the charts may be used with equal facility to find *design* and *possible capacities* in accordance with AASHO practice, or to find *maximum service volumes* for any desired level of service, A to E, in accordance with the *Manual* procedure.

Table 3.—Factor *f* for adjustment to various levels of service, 2-way facilities

Level of service	Load factor	Factor *f* when approach width W_A, in feet, is—								
		10	15	20	25	30	35	40	50	60
2-way street—no parking:										
A No backlog	0.0	0.85	0.86	0.87	0.88	0.89	0.90	0.89	0.87	0.85
B	0.1	0.90	0.91	0.91	0.92	0.92	0.92	0.93	0.92	0.91
C Design capacity	0.3	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	1.14	1.14	1.14	1.14	1.15	1.16	1.17	1.18	1.20
E Possible capacity	0.85	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
2-way street—with parking										
A No backlog	0.0	...	...	0.95	0.93	0.91	0.89	0.88	0.86	0.84
B	0.1	...	...	0.97	0.96	0.95	0.94	0.93	0.91	0.89
C Design capacity	0.3	...	...	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	...	...	1.06	1.09	1.11	1.14	1.17	1.22	1.24
E Possible capacity	0.85	...	...	1.10	1.14	1.18	1.21	1.25	1.31	1.34
Rural highway										
A No backlog	0.0	0.92	0.95	0.96	0.96	0.97	0.97	0.97	0.97	0.96
B Design capacity	0.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
C	0.3	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
D	0.7	1.21	1.21	1.22	1.23	1.25	1.27	1.29	1.31	1.33
E Possible capacity	0.85	1.28	1.28	1.28	1.30	1.32	1.35	1.38	1.41	1.44

Table 4.—Factor *f* for adjustment to various levels of service, 1-way facilities

Level of sevice	Load factor	Factor *f* when approach width W_A, in feet, is–								
		10	15	20	25	30	35	40	50	60
1-way street—no parking										
A No backlog	0.0			0.95	0.95	0.95	0.94	0.94	0.94	0.93
B	0.1			0.97	0.97	0.97	0.96	0.96	0.96	0.95
C Design capacity	0.3			1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7			1.12	1.09	1.07	1.07	1.08	1.11	1.13
E Possible capacity	0.85			1.15	1.13	1.12	1.12	1.13	1.15	1.17
1-way street—parking one side										
A No backlog	0.0			0.90	0.89	0.89	0.89	0.88	0.87	0.86
B	0.1			0.93	0.93	0.93	0.93	0.92	0.91	0.90
C Design capacity	0.3			1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7			1.07	1.08	1.10	1.12	1.14	1.17	1.22
E Possible capacity	0.85			1.10	1.13	1.16	1.18	1.20	1.25	1.30
1-way street—parking both sides:										
A No backlog	0.0				0.88	0.85	0.84	0.83	0.83	0.82
B	0.1				0.91	0.90	0.90	0.89	0.88	0.88
C Design capacity	0.3				1.00	1.00	1.00	1.00	1.00	1.00
D	0.7				1.17	1.17	1.17	1.18	1.22	1.25
E Possible capacity	0.85				1.25	1.25	1.25	1.27	1.32	1.37

Factors Affecting Intersection Capacity

The charts, together with the designated procedure, incorporate all factors affecting intersection capacity covered in the *Manual.* These factors, which are listed below, also indicate the data needed to analyze the capacity of a given intersection or to determine its required geometry for a given traffic volume, and they are accounted for in the charts either directly or in combination with supplementary charts or tables.

Basic Physical and Operating Conditions:
- One-way or two-way operation,
- Parking condition, and
- Width of approach.

Environmental Conditions:
- Metropolitan area population,
- Location within metropolitan area,
- Peak-hour factor, and
- Load factor.

Traffic Characteristics:
- Traffic volume to be served on each approach,
- Turning movements,
- Trucks and through buses, and
- Local buses (bus stops).

Control Measures:
- Traffic signals—functional type and phasing, and
- Degree of channelization and approach lane markings.

Design of Nomographs

Chart makeup

The charts presented here for the graphic solution of intersection capacities are nomographs of the stepped variety. They perform a series of multiplications and algebraic additions necessary for various adjustments to find design capacities for given roadway and traffic conditions. The first two charts are simple arrangements for determining the design capacity of one approach to a signalized intersection for average conditions. Chart 1 is for 2-way, and chart 2 for 1-way, facilities. The upper part of each nomograph is a plot of the curves from figures 1 and 2 with a load factor of 0.3 for the different types of facilities (except for rural highways), parking conditions, and location within the metropolitan area. The curves relate the approach width to design capacity for average conditions in terms of vehicles per hour of green. The lower part of the chart is a proportional graph that converts for a given signal timing the design capacity to a volume in vehicles per hour. The third graph unit on the right adjusts this volume to a given metropolitan size.

The two charts are applicable to situations where only approximate solutions are required or where specific traffic characteristics are not known. They also form the basis for developing additional nomographs for specific conditions. The upper curves, five in chart 1 and eight in chart 2, are the basis for the 13 detailed nomographs in charts 3–15. Each curve in the succeeding charts, expanded to a family of curves representing various percentages of trucks in the traffic stream, forms the upper section of a separate nomograph. The signal-timing-adjustment (G/C ratio) part of the graph in charts 1 and 2 is used to form the last section of the succeeding charts. The intermediate parts of the nomographs, charts 3–15, account successively for the effects of right turns, left turns, and metropolitan area size.

These nomographs are supplemented by: chart 16, which provides adjustments for conditions where there are bus stops at the intersection; chart 17, which determine capacities of separate right- and left-turn lanes **without** separate signal indication; and chart 18, which determines capacities of separate right- and left-turn lanes **with** separate signal indication.

The last two nomographs, charts 19 and 20 (to be published in Oct. 1967 issue), are designed for use in planning street systems and in preliminary design, or for review of plans where approximate but quick solutions are desired in terms of total or overall intersection capacity. The charts are augmented by several tables and special conditions that can be used for complete analyses of practically any form of signalized intersection problem.

Definitions and chart terminology

The factors affecting capacity, referred to in the charts, are defined in the following statements. To reduce these factors to simple terms on the charts and in the examples, a system of symbols is employed. The factors used at the outset to organize the charts are (1) 1-way or

EXAMPLE

Given

Two-way, 66-ft Street
No Parking

Metro Pop. — 400,000
Fringe Area

$W_A = 33'$

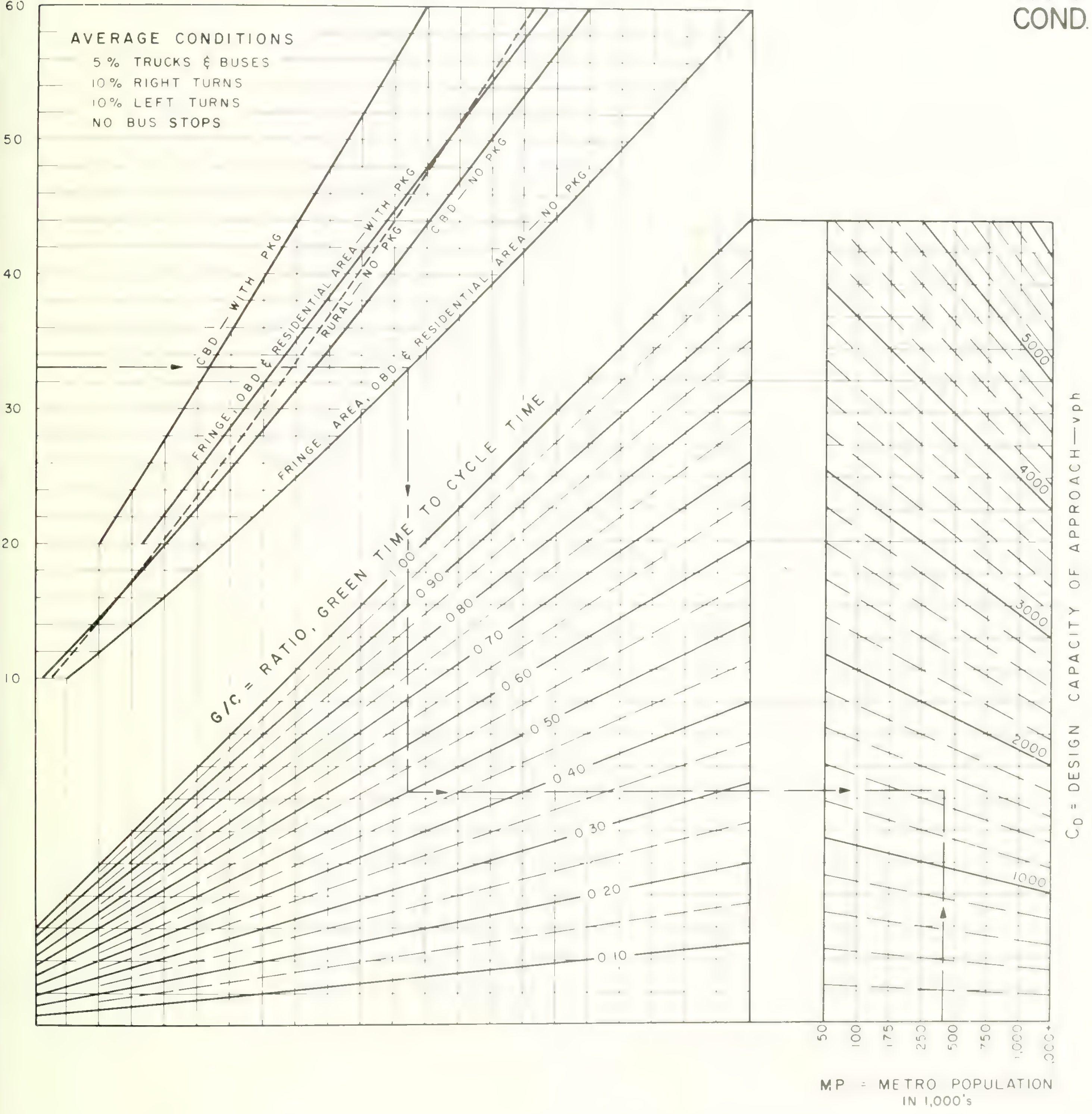
$G/C = 0.50$

Solution

$C_D = 1500$ vph

1

AVG.
COND.



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY FACILITIES — AVERAGE CONDITIONS

CHART 1

2-way operation, (2) parking conditions, and (3) location within the metropolitan area. The various combinations of the three conditions produce the 13 separate charts numbered 3–15. These conditions are.

1-way or 2-way Operation

There are major differences in the operation of 1-way and 2-way approaches which are reflected in the capacities attained. Because of this, analysis procedures for the 1-way and 2-way approaches are handled separately.

Parking Conditions

No parking (*NO PKG*) is defined as no standing and no stopping on the approach, other than an occasional passenger discharge or pickup.

With parking (*PKG*) means that vehicles are present, standing attended or unattended, along the curb on the approach. For capacity considerations, only the actual presence or absence of parked vehicles applies, not posted parking regulations.

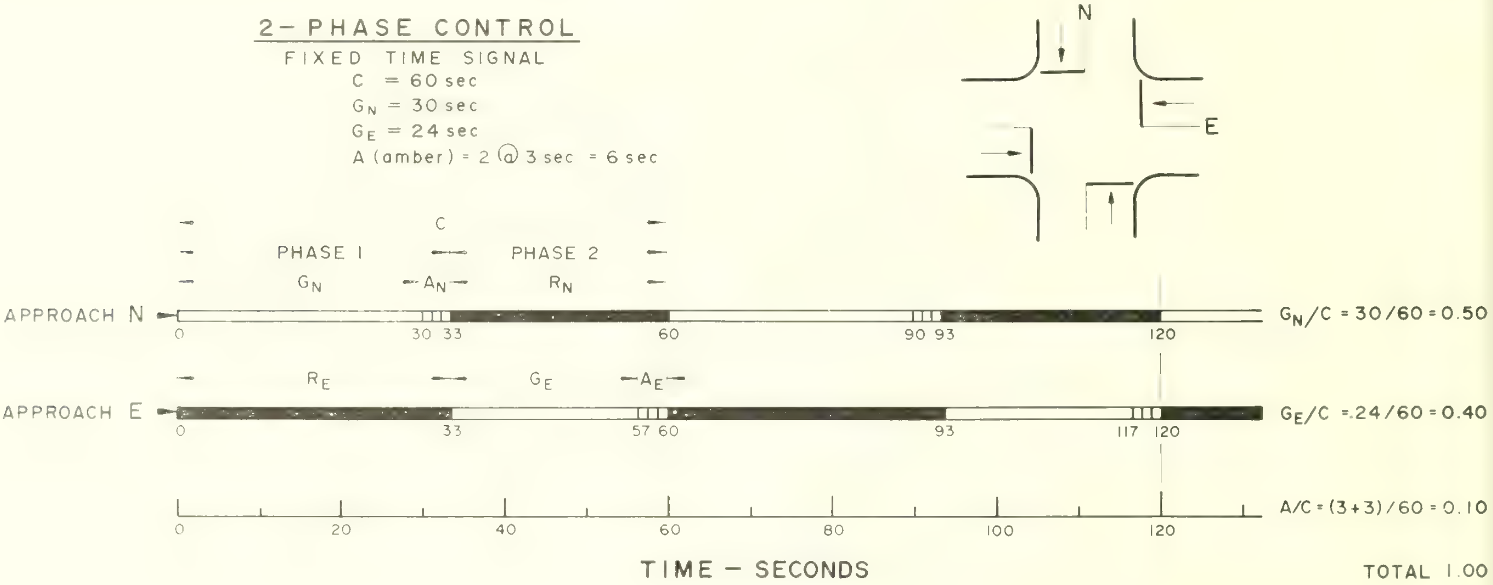


Figure 3.—Signal phasing and G/C ratio illustrated.

Location Within Metropolitan Area

Central Business District (*CBD*) is that portion of a municipality in which the dominant land use is for intense business activity.

Fringe Area (*FRNG*) is that portion of a municipality immediately outside the central business district in which there is a wide range in type of business activity, generally including small commercial, light industrial, warehousing, automobile service activities, and intermediate strip development, as well as some concentrated residential areas.

Outlying Business District (*OBD*) is that portion of a municipality or an area within the influence of a municipality, normally separated geographically by some distance from the central business district and its fringe area, in which the principal land use is for business activity.

Residential Area (*RES*) is that portion of a municipality, or an area within the influence of a municipality, in which the dominant land use is residential development, but where small business areas may be included.

Area outside city environs (*RURAL*) is characterized by open country. This area is not related to metropolitan area but is grouped here as a type of area to be considered in capacity analyses and for which a separate chart has been developed.

The other factors accounted for directly on charts 3–15 and the symbols adopted for them are noted as follows:

W_A Width of approach to the intersection, in feet. For a 1-way street, it is normally the total curb-to-curb width. For a 2-way, undivided street, it is the actual width utilized by traffic approaching the intersection, normally measured from the outer curb to the division line. For a 2-way, divided street, it is the width of the approach roadway, normally measured from the outer curb to the curb or edge of median. W_A is always exclusive of specially designated turning lanes.

T Trucks and buses, exclusive of light delivery trucks, as a percentage of the total approach volume. Local buses stopping to pick up and discharge passengers are counted along with through buses and are presented in the charts.

R Right-turning vehicles expressed as a percentage of the total volume on the approach.

L Left-turning vehicles expressed as a percentage of the total volume on the approach.

MP Metropolitan area population and the peak-hour factor both have a significant effect on intersection capacity. Several pertinent points are noted:

- Nine population groups are considered in the *Manual*, including a variety of metropolitan areas from small single cities to widespread urbanized areas composed of several cities. These are listed in table B on charts 3–14. Higher capacities are associated with the larger metropolitan areas. Application of resultant criteria to satellite communities requires judgment and/or local study to establish whether the community is better considered as a separate independent small city or an outlying part of the large central city.
- The peak-hour factor (*PHF*) is a measure of consistency of traffic demand. For intersections, it is defined as the ratio of the number of vehicles accommodated during the peak hour to four times the number of vehicles accommodated during the *highest* 15 consecutive minutes. *Higher* capacities are associated with the larger peak-hour factors.
- For convenience in the solution of problems, the effects of metropolitan area population and peak-hour factor are consolidated in a single adjustment factor (*MP*) on charts 3–14. Table B on the charts is entered first with metropolitan area population and peak-hour factor. The resultant adjustment factor is then applied in the nomograph.
- Sometimes the peak-hour factor may not be available, particularly on new facilities planned for the future. Then the application of the *MP* adjustment in the charts may be accomplished by the use of a metropolitan (metro) area population only, labelled on the curves. That is, population is used directly

2

AVG. COND.

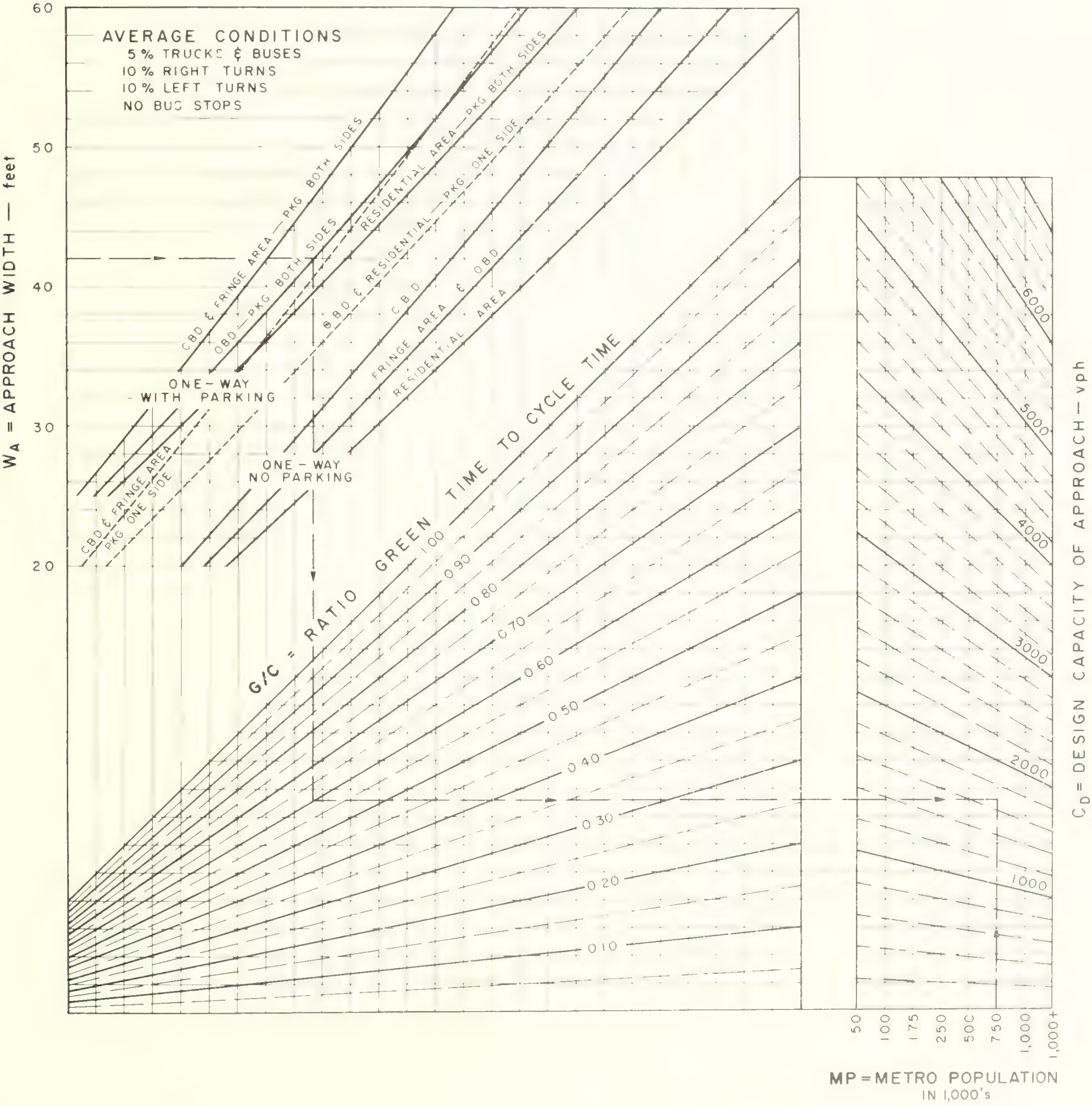
EXAMPLE

Given

One-way, 42-ft Street
Parking Both Sides
Metro Pop. — 750,000
Outlying Business District
W_A = 42′
G/C = 0.60

Solution

C_D = 1700 vph
C_P = 1700 x 1.28 = 2170 vph
(Factor 1.28 from Table 2)



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — AVERAGE CONDITIONS

CHART 2

in the chart without using table B data. The indicated population size, however, incorporates an average or representative peak-hour factor. This built-in peak-hour factor ranges from 0.80 for a metro population of 50,000 to 0.90 for a metro population of more than 1 million, based on the averages of data collected for nearly 800 signalized urban intersections.

G/C Proportion of total time during the peak hour that the signal is green for the movement of traffic from the one approach. For a fixed-time signal, in which the length of intervals of the green and red indications are held constant throughout the hour, the hourly proportion of green for a given approach is measured directly by the ratio of G to C, where G is the green interval in seconds, and C is the total cycle (including green, amber, and red intervals) in seconds. Phase sequences and the formation of the G/C ratio are illustrated in figure 3 for a standard 2-phase signal. For 3-phase control, three bars would be shown, and three green, red, and amber intervals would constitute a cycle.

The signal phasing may be altered during different periods of the day. A 3-dial control, which provides different signal phasings for the a.m. peak period, the p.m. peak period, and the off-peak periods, is a popular form of fixed-time control in urban areas. The 3-dial signal setting allows for greater efficiency in utilization of the signal by fitting more closely the demands of traffic during the several periods throughout the day. For this reason, traffic information should provide one design hour volume (DHV) for the a.m. peak period and a separate DHV for the p.m. peak period. Additional peaks may be considered, such as those occurring during summer weekends, heavy truck operations, etc.

The G/C principle for analysis is also applicable, with some modification, to actuated control and to progressive signal systems. Procedures for other than fixed-time control are covered in part 3 under the heading *Signal Systems Other Than Fixed Time.*

PART 1—2-WAY FACILITIES

Intersections With Average Conditions

Capacity analyses for planning and preliminary design stages generally are accomplished in simplified form. As specific conditions relating to turning movements and truck percentages are not known under such circumstances, procedures using average conditions are appropriate. Chart 1 allows the determination of design capacities for conditions with and without parking for different locations within metropolitan areas of different sizes and for conditions within rural areas. Average conditions constitute 5-percent trucks and buses, 10-percent right turns, 10-percent left turns, and no bus stops.

Problem 1

What is the design capacity of a 2-way street, 66 feet wide curb-to-curb, with parking prohibited, in a fringe area within a metropolitan (Metro) size of 400,000 population? Major intersections are signalized. Specific data regarding commercial vehicles, turning movements, etc., are not known, but conditions are assumed to be average. Half the time during the hour can be allotted to green on this street.

Solution: Using $W_A=66/2=33$ and $G/C=0.50$, and following the arrows indicated in chart 1, it is determined that design capacity $C_D=1{,}500$ v.p.h. in one direction. If parking were permitted, C_D would be 1,070 v.p.h.

Problem 2

A major street consisting of a narrow median and two 21-foot traveled ways with no parking, in a residential area within a metro size of 100,000 population, is to have a signal installed at a cross street. Conditions are assumed to be average. If the cross street requires a G/C of 0.35, what would be the design capacity of the major street? If the approach volume, DHV, is 1,350 v.p.h., what width of traveled way is required in each direction, without exceeding design capacity, assuming a lane width of not less than 11 feet?

Solution: Normally the amber periods, for purposes of capacity analyses, are considered to be 10% of the cycle. Therefore, G/C available for the major street approaches is $1.00-0.35-0.10=0.55$. Proceeding through chart 1 with $W_A=21$, residential area without parking, $G/C=0.55$, and $MP=100{,}000$, a value of $C_D=910$ is found.

To handle an approach volume of 1350 v.p.h., the required width of approach is found by proceeding through the chart in reverse order. Proceeding through the chart with $MP=100{,}000$, $C_D=1{,}350$, $G/C=0.55$, and residential area without parking, $W_A=31$ feet is found. Using 11-foot lanes, the required width of approach is 33 feet.

Problem 3

In a central business district of a 250,000 population metro area, a 2-way 58-foot street with parking intersects a 2-way 44-foot street with no parking. The former is to accommodate a peak-hour volume of 620 v.p.h. in one direction. If conditions are assumed to be average and a 60-second cycle is used, 6 seconds of which are allotted to amber, what should be the green interval on the 58-foot street for operation at design capacity? What would be the resultant green interval and design capacity of one approach on the 44-foot street? What would be the possible capacity of this approach?

Solution: Enter chart 1 at left with $W_A=58/2=29$, proceed right to CBD with parking curve, then down to lower graph until a horizontal projection of 620 v.p.h. is intersected; $G/C=0.43$. G on 58-foot street$=60\times0.43=26$ seconds. G on 44-foot street$=60-26-6=28$ seconds; and $G/C=28/60=0.47$.

For the design capacity of the 44-foot street using $W_A=44/2=22$, CBD without parking, and $G/C=0.47$, in chart 1, $C_D=730$ v.p.h. in one direction.

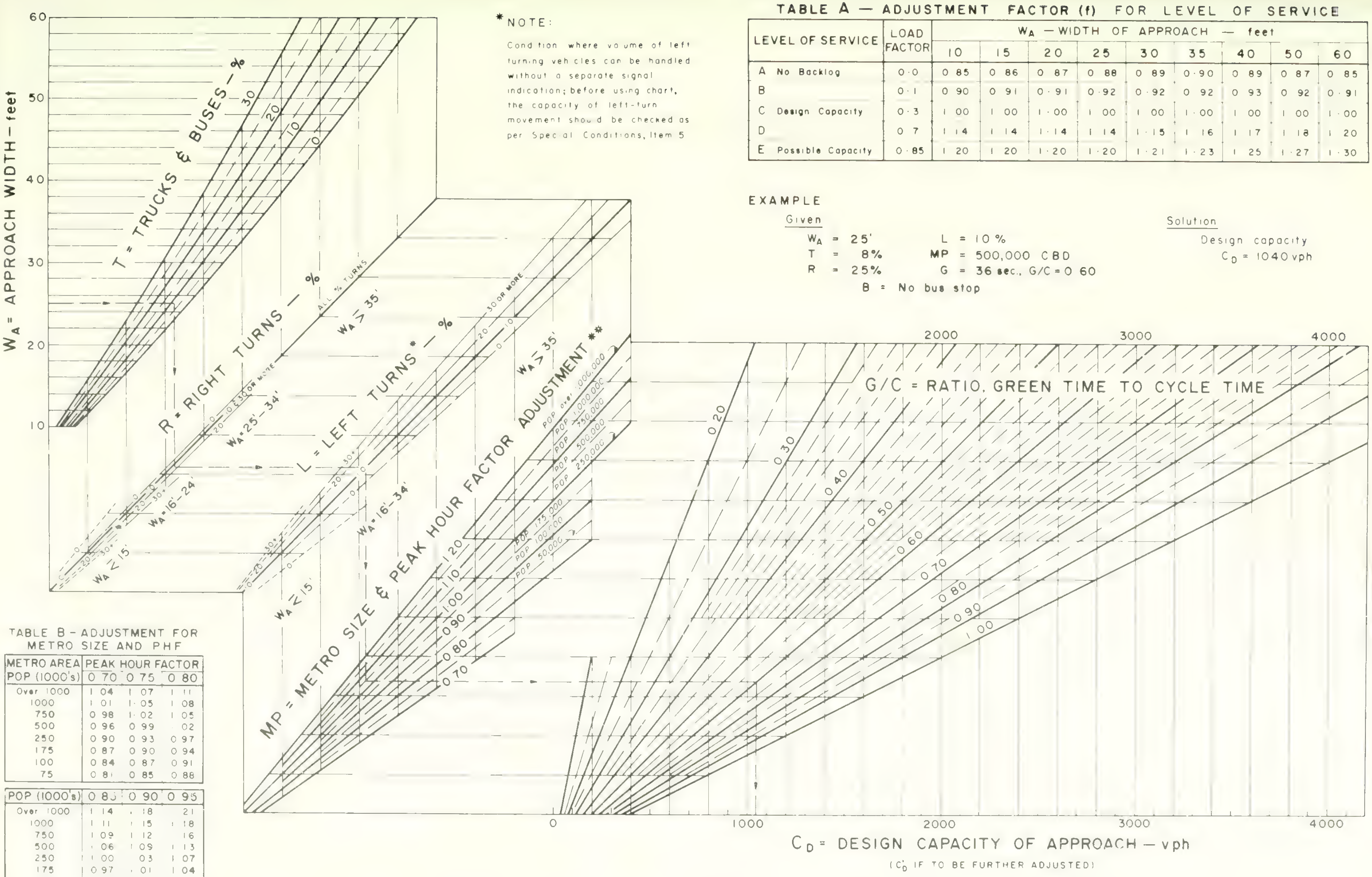
For the possible capacity of the 44-foot street, using $f=1.2$ from table 2 for a 22-foot approach with no parking, $C_P=730\times1.2=880$ v.p.h. in one direction.

Streets Without Parking

Charts 3 and 4 include the adjustments for specific intersection conditions on 2-way streets where parking is prohibited. Chart 3 is applicable to central business districts, and chart 4 to all other sections within the metropolitan area—fringe, outlying business, and residential. Charts 3 and 4 will often be used by proceeding from the left side to bottom scales, as shown by arrows. The charts can also be used in the reverse order to obtain the width of approach required to handle a given volume. Or, with a given approach volume and a given width, the necessary ratio of green time to cycle time can be determined readily. When the approach width and the G/C ratio cannot be altered, but capacity must be increased, the increase through elimination of left turns and/or changing the bus-stop

3

C.B.D.



*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	0.85	0.86	0.87	0.88	0.89	0.90	0.89	0.87	0.85
B	0.1	0.90	0.91	0.91	0.92	0.92	0.92	0.93	0.92	0.91
C Design Capacity	0.3	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	1.14	1.14	1.14	1.14	1.15	1.16	1.17	1.18	1.20
E Possible Capacity	0.85	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30

EXAMPLE

Given

W_A = 25'
T = 8%
R = 25%
L = 10%
MP = 500,000 CBD
G = 36 sec., G/C = 0.60
B = No bus stop

Solution

Design capacity
C_D = 1040 vph

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	1.04	1.07	1.11
1000	1.01	1.05	1.08
750	0.98	1.02	1.05
500	0.96	0.99	1.02
250	0.90	0.93	0.97
175	0.87	0.90	0.94
100	0.84	0.87	0.91
75	0.81	0.85	0.88

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.18	1.21
1000	1.11	1.15	1.18
750	1.09	1.12	1.16
500	1.06	1.09	1.13
250	1.00	1.03	1.07
175	0.97	1.01	1.04
100	0.94	0.98	1.01
75	0.92	0.95	0.98

** Use Table B if PHF is known to find adjust factor, otherwise use Population directly

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — NO PARKING — C.B.D.

CHART 3

condition with the aid of chart 16, can be found on the charts.

Charts 3 and 4 apply to intersection approaches that have the conditions described on the charts. The direct use of the charts as indicated by example arrows, however, is applicable specifically to the condition in which the volume of left-turning vehicles on the approach can be handled without requiring a separate signal indication. A check for the capacity of left-turn movement should always be made when using charts 3 and 4 and also charts 5–15, as explained in *Part 3, Special Conditions* under the heading *Check for Capacity of Left Turn*. For simplicity in demonstration and better understanding of chart use, examples 4–8 are purposely selected so that the volume of left-turning vehicles does not exceed the capacity of the left-turn movement. The maximum service volume of left-turning vehicles that can be accommodated without a separate signal indication on major streets is generally in the range of 80 to 120 v.p.h., and a left turning volume of 100 v.p.h. can be used as an average. Further examples illustrate the use of this important control.

Where bus stops are provided at the intersection for pickup and discharge of passengers, design capacity values obtained from charts 3 and 4 are corrected by multiplying the result by an appropriate adjustment factor from chart 16. When the charts are used in reverse order, to find G/C or W_A, C_D is divided by the bus adjustment factor before the charts are entered.

Where levels of service other than C (design capacity) are desired, including level of service E (possible capacity), the resultant value of C_D in charts 3 and 4 should be multiplied by an appropriate adjustment factor from table A shown at upper right on charts 3 and 4. When the charts are used in reverse order, C_D is divided by the level of service factor before the charts are entered.

Problem 4

Determine the design capacity of one approach on a 50-foot, 2-way street without parking, located in the *CBD* of a metro area with a population of 500,000. Other conditions at the intersection are: $T=8\%$, $R=25\%$, $L=10\%$, $G=36$ seconds, $C=60$ seconds, and no bus stop.

Solution: Enter chart 3 with $W_A=25$ and follow the arrows according to each condition; find $C_D=1{,}040$ v.p.h.

Problem 5

Determine the design capacity and possible capacity of one approach on a 64-foot 2-way street without parking, where the cycle is 60 seconds and green interval is 27 seconds. The intersection is located in the *CBD* of a metro area with a population of 250,000. Other conditions are: $T=12\%$, $R=15\%$, $L=7\%$, and bus stop on near side serving 46 buses per hour.

Solution: Enter chart 3 at left with $W_A=32$ and using $T=12\%$, $R=15\%$, $L=7\%$, $MP=$ pop. 250,000, and $G/C=27/60=0.45$; find $C'_D=930$. Using chart 16—no parking, near side $B=46$ and $W_A=32$ (see arrows on chart) find adjustment factor, $F_B=0.81$. Design capacity $C_D=930\times 0.81=750$ v.p.h., in one direction.

From table A in chart 3, for possible capacity and $W_A=32$, $f=1.22$. Possible capacity $C_P=750\times 1.22=920$ v.p.h. in one direction.

Problem 6

A 2-way street on which there will be no parking is planned to cross an existing street. The facility is located in the *CBD* of a metro area having a population of 250,000 and a peak-hour factor (*PHF*) of 0.80. According to the volume on the existing street, 33% of the cycle time must be allotted to green on that street. Determine the needed width of pavement on the new street if the *DHV* in one direction is 1,200 v.p.h. and other conditions are: $T=14\%$, $R=12\%$, $L=5\%$, no bus stop, and C is to be set at 70 seconds with an allowance of 6 seconds for amber per cycle.

Solution: The time available for the amber periods and for the green on the new street is $1.00-0.33=0.67$. Therefore, $(G+\text{amber})\div C=0.67$, or $(G+6)\div 70=0.67$; $G=41$ seconds, and $G/C=0.59$.

Enter chart 3 at bottom with a peak-hour volume of 1,200 v.p.h. in one direction, and proceed through the chart, turning at $G/C=0.59$, $MP=0.97$ (determined from table B), $L=5\%$, $R=12\%$, and $T=14\%$; find $W_A=31.5$ feet. In proceeding through the chart, it was necessary to assume for the L and R turning points that either $W_A=(16 \text{ to } 34)$ or $W_A \overline{>} 35$. If the proper assumption is made initially, the answer is determined directly. If the assumption is incorrect, a second trial is required to determine the correct answer. If 11-foot lanes are to be used, and operation is generally balanced by direction, the new street should be the nearest multiple doubled for both directions; $33\times 2=66$ feet wide.

Problem 7

Determine the design capacity of an intersection approach on a street with a narrow median and two 30-foot pavements in the fringe area of a city having a metro population of 700,000 and *PHF* of 0.89. Other conditions are: $T=9\%$, $R=14\%$, left turns prohibited, $G=31$ seconds, $C=65$ seconds, and a bus stop on the far side serving 70 buses per hour.

Solution: As a preliminary step, find $MP=1.11$ by interpolation in table B, chart 4; $G/C=31/65=0.48$; and $F_B=0.95$ in chart 16—no parking, far side—using 70 buses, $W_A=30$ and $(L+R)=14\%$. Proceed through chart 4 from upper left with $W_A=30$ and follow the arrows according to each condition, as illustrated on the chart. Find initial $C'_D=1{,}460$; with adjustment for local buses, design capacity, C_D, is $1{,}460\times 0.95=1{,}390$ v.p.h. in one direction.

Problem 8

In a residential area of a city with a metro population of 1 million, a 2-way, 40-foot parkway is to be crossed by a new street. On the critical approach of the existing parkway the projected peak-hour traffic volume in one direction is 900 automobiles of which 135 turn right and 80 turn left. On the critical approach of the new street, the design volume is 1,680 v.p.h. of which 7% are trucks; turning movements are 10% and 4% to the right and left, respectively. There will be no parking and no bus stops at the intersection on either facility. If 12-foot lanes are to be used, how many lanes are required on one approach of the new facility?

Solution: It is first necessary to determine the proportion of green time required for the existing parkway. From chart 4, using $W_A=20$, $T=0\%$, $R=135/900=15\%$, $L=80/900=9\%$, no bus stops, and intersecting from a volume (equal to C_D) of 900 v.p.h., it is found that $G/C=0.43$.

Assume $C=60$ seconds and total amber period is 6 seconds. Then, G for parkway traffic is $0.43\times 60=26$ seconds, and G for traffic on the new highway is $60-26-6=28$ seconds.

To determine the required width of the new facility, enter the bottom of chart 4 at a design volume of 1,680 v.p.h., and proceed up and to the left using $G/C=28/60=0.47$, $MP=$ pop. 1,000,000, $L=4\%$, $R=10\%$, and $T=7\%$; find $W_A=35.5$ feet. Three 12-foot lanes, therefore, are required on the new highway in each direction of travel.

Streets With Parking

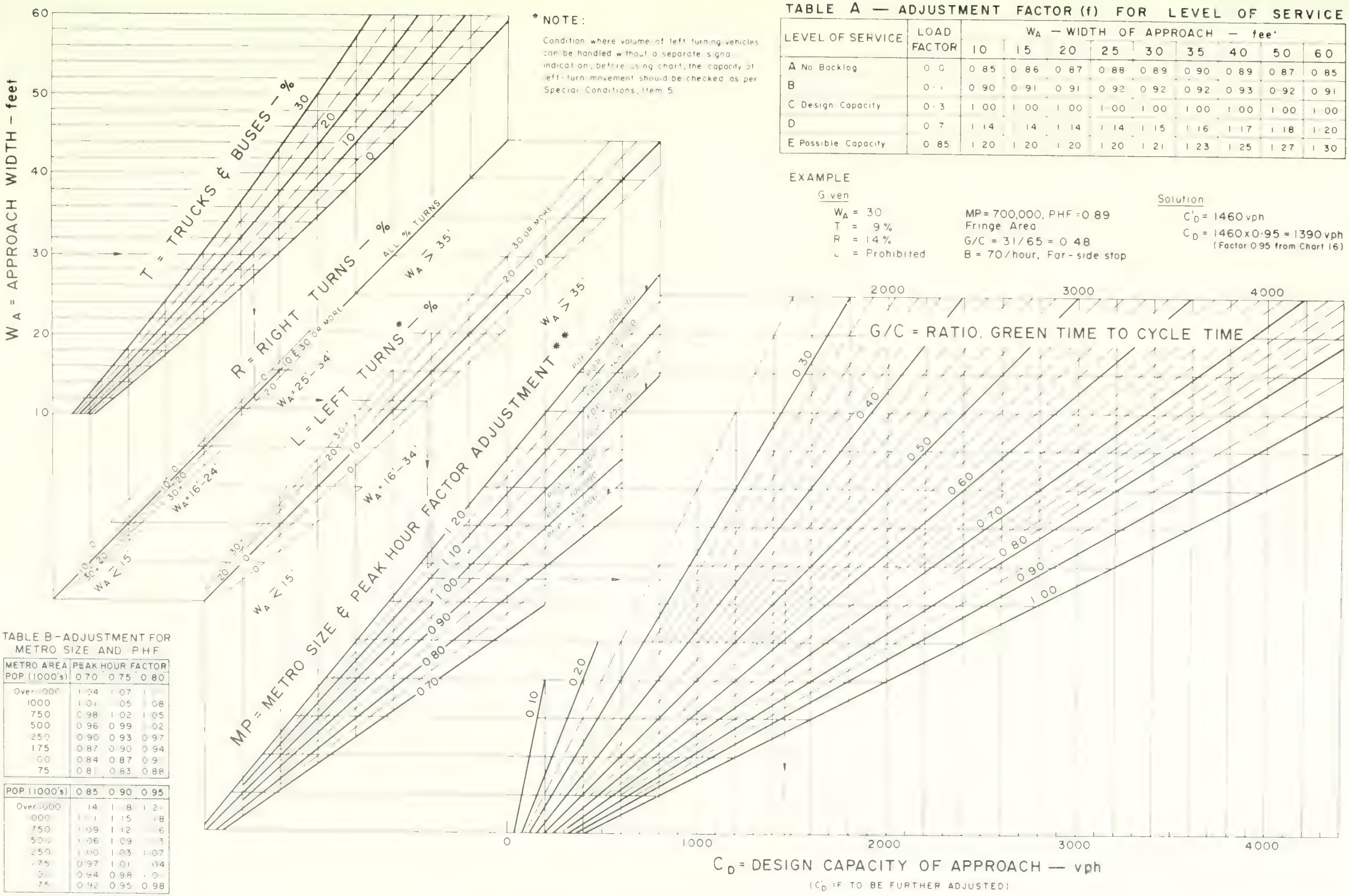
Charts 5 and 6 include the adjustments for specific intersection conditions on 2-way streets with parking permitted. (In this article, as in the *Manual*, parking parallel to the curb is the only type considered; diagonal parking would obviously have a much different effect on traffic flow.) Chart 5 is applicable to central business districts, and chart 6 to all other sections within the metropolitan area—fringe, outlying business and residential.

The procedure through the charts is the same as for charts 3 and 4 as previously described. Also, the direct use of charts 5 and 6 applies to the condition where the volume of left-turning vehicles on the approach can be accommodated without requiring a separate signal indication. For simplicity in presentation, examples 9–13 were selected so that this condition is satisfied, as it would be ordinarily when the volume of left-turning vehicles on one approach does not exceed 100 v.p.h. Adjustments for bus stops, in accordance with chart 16, and for levels of service other than C, in accordance with table *A*, shown at upper right of charts 5 and 6, are accomplished in the same way as for charts 3 and 4.

Problem 9

Determine the design capacity and possible capacity of one approach on a 2-way, 60-foot street, with parking in the central business district of a metro area having a population of 750,000. Other conditions are: $T=8\%$, $R=14\%$, $L=0\%$ (left turns prohibited), $G=34$ seconds, $C=65$ seconds, and a far-side bus stop that serves 70 buses during the peak-hour period.

Solution: Enter chart 5 at left with $W_A=30$ and proceed through the chart using the conditions listed. The initial result, as shown by the arrows in the chart, is a design capacity C'_D of 900 v.p.h.



*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	0.85	0.86	0.87	0.88	0.89	0.90	0.89	0.87	0.85
B	0.1	0.90	0.91	0.91	0.92	0.92	0.92	0.93	0.92	0.91
C Design Capacity	0.3	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	1.14	1.14	1.14	1.14	1.15	1.16	1.17	1.18	1.20
E Possible Capacity	0.85	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30

EXAMPLE

Given

W_A = 30
T = 9%
R = 14%
L = Prohibited

MP = 700,000, PHF = 0.89
Fringe Area
G/C = 31/65 = 0.48
B = 70/hour, Far-side stop

Solution

C'_D = 1460 vph
C_D = 1460 x 0.95 = 1390 vph
(Factor 0.95 from Chart 16)

TABLE B–ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	1.04	1.07	[illegible]
1000	1.01	1.05	1.08
750	0.98	1.02	1.05
500	0.96	0.99	1.02
250	0.90	0.93	0.97
175	0.87	0.90	0.94
100	0.84	0.87	0.9[illegible]
75	0.8[illegible]	0.83	0.88

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.18	1.2[illegible]
1000	[illegible]	1.15	1.18
750	1.09	1.12	[illegible]6
500	1.06	1.09	[illegible]3
250	1.00	1.03	1.07
175	0.97	1.01	1.04
100	0.94	0.98	1.0[illegible]
75	0.92	0.95	0.98

** Use Table B [illegible]

C_D = DESIGN CAPACITY OF APPROACH — vph
(C'_D IF TO BE FURTHER ADJUSTED)

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — NO PARKING — O.B.D., FRINGE & RESIDENTIAL AREAS

CHART 4

FRNG.
O.B.D.
RES.

4

Because there is a bus stop on the far side, with parking, the lower part of chart 16-B is used; for 70 buses, *CBD*, $W_A=30$ feet, and $(L+R)=14\%$, $F_B=0.98$.

Design capacity, adjusted for the bus stop, is $C_D=900\times0.98=880$ v.p.h. Adjustment factor for possible capacity for a 30-foot approach is $f=1.18$ (table A on chart 5); $C_P=880\times1.18=1{,}040$ v.p.h.

Problem 10

Determine the design capacity of a 32-foot approach of a 2-way street with parking in an outlying business district of a metro area having a population of 400,000. Other conditions: $T=5\%$, $R=16\%$, $L=7\%$, $G=36$ seconds, $C=62$ seconds, and a near-side bus stop handling 50 buses during the peak-hour period.

Solution: Using $W_A=32$, and following the arrows in chart 6 for the conditions indicated, $C'_D=1{,}220$ v.p.h.

Adjustment for buses is found in the upper part of chart 16-B. Using $B=50$, outlying-area turning line and $(L+R)=23\%$, in conjunction with $W_A=24$ and $W_A=36$, an adjustment of $F_B=1.06$ and 1.08, respectively, is found. Interpolating for $W_A=32$, $F_B=1.07$.

Design capacity of one approach, $C_D=1{,}220\times1.07=1{,}310$ v.p.h.

Problem 11

As shown in figure 4, an east-west 52-foot, 2-way street operates with parking on the south side only. Two lanes are available for moving traffic on both the west and east approaches. The intersection is located in the fringe area of a city having a metro population of 200,000. If the $PHF=0.75$ and the evening peak-hour traffic is as indicated in figure 4, determine the signal timing required on the east-west street to accommodate this traffic, using a 60-second cycle.

Solution: For the west approach, as there is parking, chart 6 is applicable. The fact that there is no parking on the north side of the street or on the east approach does not affect the operation or the analysis of the west approach, which does have parking. Enter chart 6 at upper left with $W_A=31$ feet and proceed through the chart using $T=12\%$, $L=40/660=6\%$, $R=60/660=9\%$, and $MP=0.89$ (adjustment factor interpolated from table B for metro population of 200,000 and $PHF=0.75$). The horizontal projection from the *MP* turning line is then intersected by a vertical projection from a C_D value on the bottom scale equal to the approach volume of 660 v.p.h.; find $G/C=0.40$.

For the east approach, as there is no parking, chart 4 is applicable. Enter chart with $W_A=21$ feet and, using $T=9\%$, $R=50/810=6\%$, $L=80/810=10\%$, and $MP=0.89$, find the point of intersection on the G/C scale for C_D equal to an approach volume of 810 v.p.h.; $G/C=0.50$.

The required signal timing for the east-west street, therefore, is the larger of the two G/C values or 0.50, and a G of $0.50\times60=30$ seconds.

Rural Conditions

Capacities and service volumes representative of different levels of service for specific intersection conditions in rural areas may be determined by using chart 15. The general format of the nomograph and procedure for application of the nomograph is the same as for charts 3–6. The fourth set of turning lines in chart 15 differs from a similar set in charts 3–6 because only the peak-hour factor is accounted for in chart 15 as population is irrelevant. Normally, all rural conditions are represented by a peak-hour factor of 0.70, for which the adjustment factor in the chart is 1.00.

On recreational routes operating under peak-flow conditions that build up to a *PHF* of 1.00, the capacities and service volumes according to the *Manual* may be adjusted upward by 40%. Thus an adjustment factor of 1.40 is indicated in the chart for *recreational route—high-peak flow*. An intermediate condition, referred to as *recreational route—peak flow* is introduced in the chart assuming a *PHF* of 0.85, with a corresponding adjustment factor of 1.20.

Problem 12

Determine the design capacity and possible capacity of an intersection approach of a 2-way 4-lane highway in a rural area operating under normal peaking characteristics. Other conditions are: lane width = 11 feet, $T=15\%$, $R=12\%$, $L=7\%$, and available $G/C=0.55$.

Solution: Enter chart 15 with $W_A=22$ feet and proceed through the chart, using the conditions given, as indicated by the arrows; find $C_D=740$ v.p.h. $C_P=740\times1.29=950$ v.p.h.

Problem 13

The capacities indicated in the above problem are representative of normal conditions. If the same facility during the summer weekends carries recreational traffic, what would be the design capacity and possible capacity during peak flows on these weekends? Controlling conditions are the same except that truck traffic is negligible, and right and left turns are reduced to 6% and 4%, respectively.

Solution: Enter chart 15 with $W_A=22$ and proceed through the chart turning at $T=0\%$, $R=6\%$, $L=4\%$, $PHF=0.85$ and $G/C=0.55$; find $C_D=1{,}100$ v.p.h., and $C_P=1{,}100\times1.29=1{,}420$ v.p.h.

Problem 14

If the facility in Problem 13 was expected to handle traffic from a much expanded resort area under conditions of extremely high-peak flows, approaching a peak-hour factor of 1.00, what would be the maximum 1-way volume that could be accommodated at such times? All other conditions are assumed to be the same.

Solution: Proceed through the chart as in Problem 13 but turn on $PHF=1.00$; find $C_P=1{,}280\times1.29=1{,}650$ v.p.h. in one direction.

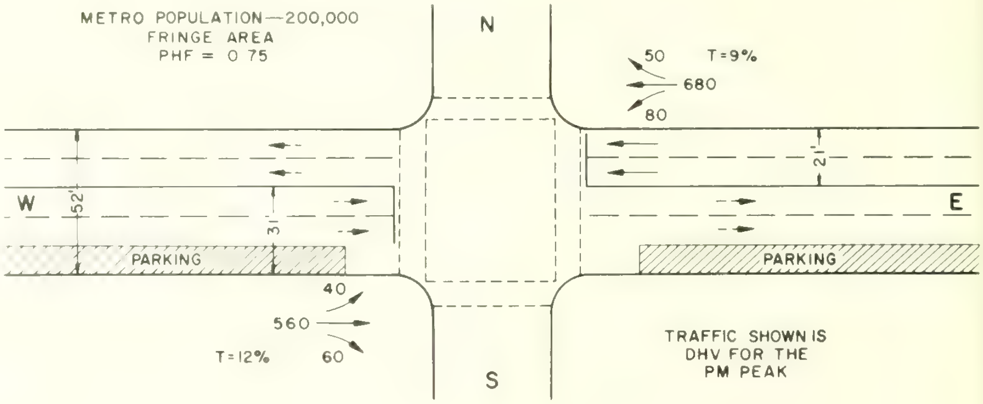
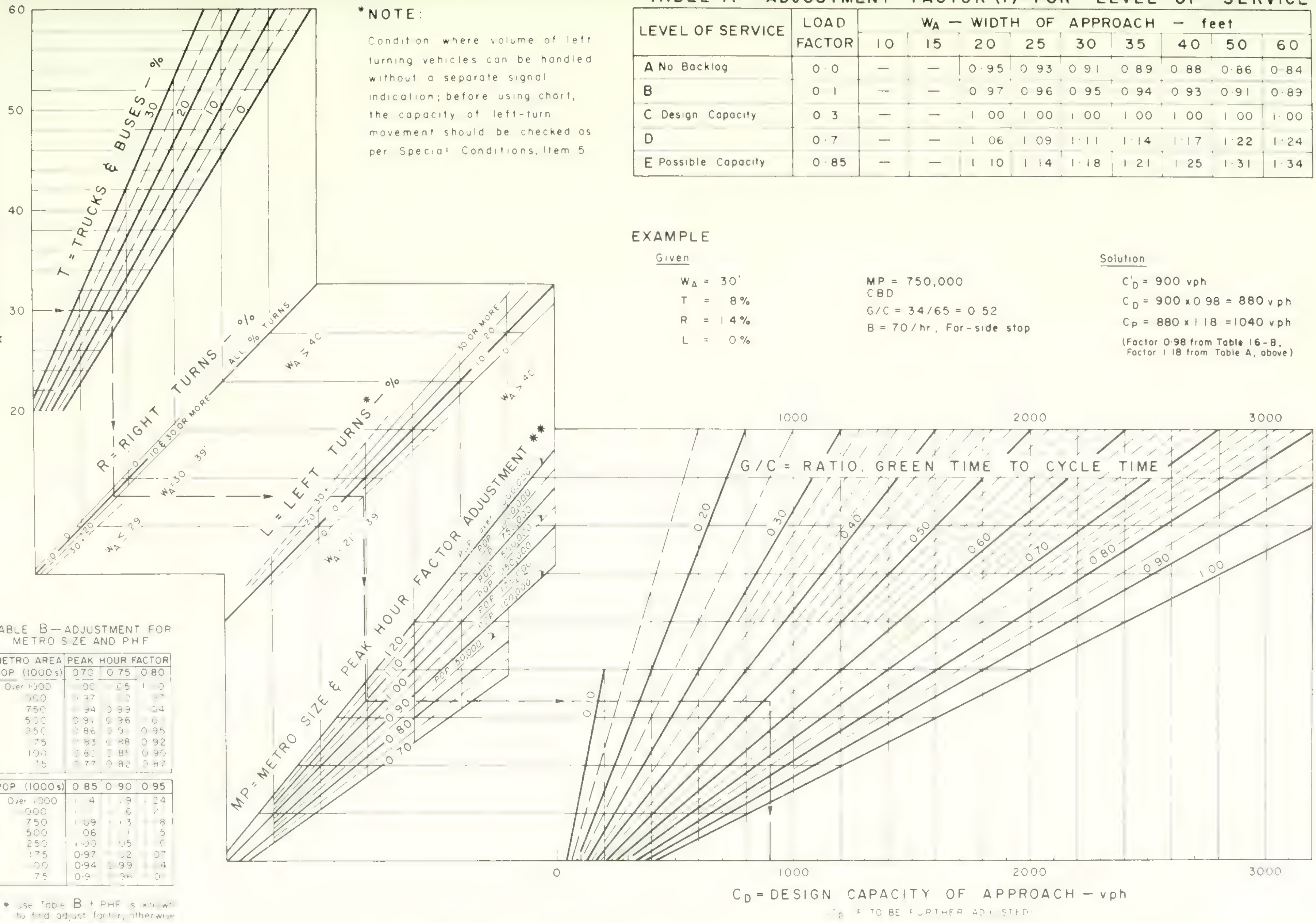


Figure 4.—Problem 11 illustrated.

Intersections With Separate Turning Lanes

Frequently, on modern highways and streets, exclusive lanes on intersection approaches are provided for left- or right-turning movements. Such lanes may be in addition to the regular width, or they may be specifically marked for turns within the basic approach width. Exclusive turning lanes may be controlled by separate signal indications, or they may be handled without additional signal phases.

Capacities of various forms of turning lanes are given in charts 17 and 18. Procedures for analyzing intersection approaches with turning lanes, including illustrative problems, are presented below. Traffic carrying capabilities are expressed in terms of design capacity and possible capacity. No distinction normally is made for other (intermediate) levels of service,



*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet 10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·93	0·91	0·89	0·88	0·86	0·84
B	0·1	—	—	0·97	0·96	0·95	0·94	0·93	0·91	0·89
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·06	1·09	1·11	1·14	1·17	1·22	1·24
E Possible Capacity	0·85	—	—	1·10	1·14	1·18	1·21	1·25	1·31	1·34

EXAMPLE

Given

W_A = 30'
T = 8%
R = 14%
L = 0%

MP = 750,000
CBD
G/C = 34/65 = 0·52
B = 70/hr, Far-side stop

Solution

C'_D = 900 vph
C_D = 900 x 0·98 = 880 vph
C_P = 880 x 1·18 = 1040 vph
(Factor 0·98 from Table 16-B, Factor 1·18 from Table A, above)

TABLE B—ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000s)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·10
1000	0·97	[illegible]	[illegible]
750	0·94	0·99	1·04
500	0·91	0·96	[illegible]
250	0·86	0·91	0·95
175	0·83	0·88	0·92
100	0·80	0·85	0·90
75	0·77	0·82	0·87

POP (1000s)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	[illegible]	[illegible]	[illegible]
750	1·09	1·13	1·18
500	1·06	[illegible]	1·15
250	1·00	1·05	[illegible]
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	[illegible]	0·96	[illegible]

** Use Table B if PHF is known to find adjust factor, otherwise use Population directly

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH PARKING — C.B.D.
CHART 5

5
C.B.D.

because data is scarce on the operation of such facilities; however, approximate relations to several other levels of service are indicated in table 5.

The capacities of turning lanes presented in charts 17 and 18 are applicable to all types of intersections, involving 1-way streets, 2-way streets, rural highways, and high-type facilities.

Table 5.—Levels of service conversion factors for separate turning lanes

Level of service	Adjustment factor
A and B	0.90
C	1.00
D	1.20
E	1.30

Turning lanes without separate signal indication

Capacities of exclusive left- and right-turn lanes where no separate signal indication is used for the turning movement are covered in chart 17. The usual form of a separate turning lane is depicted by the sketch in chart 17–A for the left-turn lane, and by the sketches in charts 17–C and 17–D for the right-turn lane. The same capacity relations apply to the form of turning lane that is part of the basic approach width.

LEFT-TURN LANE.—The design capacity of an exclusively left-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication, is given in charts 17–A and 17–B. Additional terms introduced in these charts are as follows:

D_3 Effective length of left-turn lane, in feet, for the storage of turning vehicles, exclusive of crosswalk and taper.

V_3 Volume of traffic turning left on one approach, in vehicles per hour.

T_3 Trucks and buses turning left, expressed as a percentage of the total left-turning volume V_3 on one approach.

V_o Volume of through traffic on the opposite approach, in vehicles per hour, that is in direct conflict during the same period of time with the left-turning movement on the approach in question.

T_o Trucks and buses, expressed as a percentage of the total through volume V_o on the opposite approach.

C_{D3} Design capacity of the separate left-turning lane in vehicles per hour.

The capacity of a left-turn lane is determined primarily by the volume of traffic opposing the left turn during the green signal indication. Normally, on major streets in downtown areas and on wide major streets in outlying areas, it seldom will be possible for more than two vehicles to turn left per cycle (such turns usually have to be made on the amber signal). For design purposes, between one and two left-turning vehicles accommodated per cycle are assumed to be appropriate. Design capacity of a left-turn lane, for the conditions described, is predicated on 1.6 vehicles per cycle. Chart 17-B gives this relation in terms of vehicles per hour as dependent upon the length of cycle.

On some streets, where the opposing through volume is relatively light, the capacity of a left-turn lane may be much greater than indicated above. For such a condition, the design capacity of a left-turn lane per hour of green is estimated as the difference between 1,200 and V_o, both figures expressed in terms of passenger vehicles. Chart 17-A provides a solution for this condition, being the relation between V_o, G/C, and design capacity C_{D3}. To express the capacity in terms of vehicles of all types, factors for the percentage of trucks and buses in the opposing through movement T_o and that in the left-turn movement T_3 are applied. To determine the design capacity of a left-turn lane, C_{D3} should be found on both charts 17-A and 17-B, and the larger of the two results used. Usually on major streets, the values from chart 17-B will govern. The possible capacity of a left-turn lane, whether determined by chart 17-A or 17-B, is 1.3 times the design capacity.

Lane length is another control of capacity of an added turning lane. If the lane is not long enough to store the vehicles that can make the turn on the proper green interval, the capacity otherwise possible cannot be attained. Chart 17-E gives the length of added lane needed to accommodate given volumes for different signal timings. With the chart in this form, the lane length can be determined both for capacity volumes and for known smaller turning volumes for a specific condition. Since control values are in terms of passenger vehicles only, the adjustment for percentage of trucks and buses is included.

The required (minimum) length of the added left-turn lane is determined as the distance needed to store 1.5 times the average number of turning vehicles that will accumulate per cycle, recognizing the maximum number that actually can move during the green interval. Occasionally, the maximum number of vehicles that could be stored during some cycles could be twice the average number owing to variations in arrival rates. A desirable length of turning lane, therefore, is also indicated on the chart and is predicated on two times the average number of *storing* vehicles, or a length 33% greater than the minimum dimension shown. A length of 25 feet is used for each passenger vehicle, and 40 feet for each truck or bus.

The minimum length applies to the full width of the turning lane. This full length is not available for use unless it is preceded by a suitable taper. For normal street conditions, a taper length of 70 to 100 feet may be considered appropriate; for high-type urban facilities and rural highways, it should be 150 to 300 feet. This taper is in addition to the minimum length of turning lane shown in Chart 17-E.

To determine the capacity of an intersection approach with a separate left-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication, the following procedure should be followed:

(1) Obtain C_{D3} from chart 17-A or 17-B.

(2) Obtain C_D of combined through and right-turning movement from charts 3–6 or 15, using:

W_A = approach width exclusive of left-turn lane, and
L = 0%.

(3) Obtain D_3 from chart 17-E for V_3 or C_{D3}, whichever governs.

(4) Determine possible capacity, when required:

Left-turn lane, $C_{P3} = 1.3 \times C_{D3}$.
Through and right-turning movement, combined, $C_P = f \times C_D$.
Values for f are given in table A on charts 3–6 or 15.

In this procedure, the design capacity of the intersection approach cannot be taken as the sum of separate values for the capacity of the left-turn lane and that of the through-plus-right lanes because traffic on an intersection approach follows a given pattern, or maintains a consistent proportion of the different movements—through, left, and right. Thus, the capacity of the left-turn lane and that of the through-plus-right lanes should be kept separate and, in the analysis, the volumes of the individual movements should be compared with their capacities.

Usually, the through-plus-right movement determines the G/C on the approach, and the left-turn volume is then checked for capacity of the left-turn lane based on this G/C. If the latter movement is less than design capacity, the requirements are met. However, if the left-turning volume exceeds the design capacity of the left-turn lane, the following alternatives may be considered: (1) accept a lower level of service on the left-turn lane, not to exceed possible capacity; (2) increase the G/C to provide operation of the left-turn lane at design capacity, in which case the through-plus-right volume would be less than design capacity and would operate at a higher level of service; or (3) change the signal phasing to include a separate signal indication for the left-turning movement in order to accommodate it at design capacity.

Problem 15

Determine the G/C required, during the p.m. peak period, to handle traffic without exceeding design capacity on the east approach of the intersection shown in figure 5.

Solution: Signal timing based on a combined through-plus-right volume is obtained from chart 4 using $W_A = 20$, $T = (30+65) \div (250+750) = 10\%$, $R = 250 \div 1000 = 25\%$, $L = 0\%$ (because left turn is on separate lane), $MP =$ 500,000 population, and approach volume (equated to C_D) $= 250 + 750 = 1{,}000$ v.p.h.; find $G/C = 0.57$.

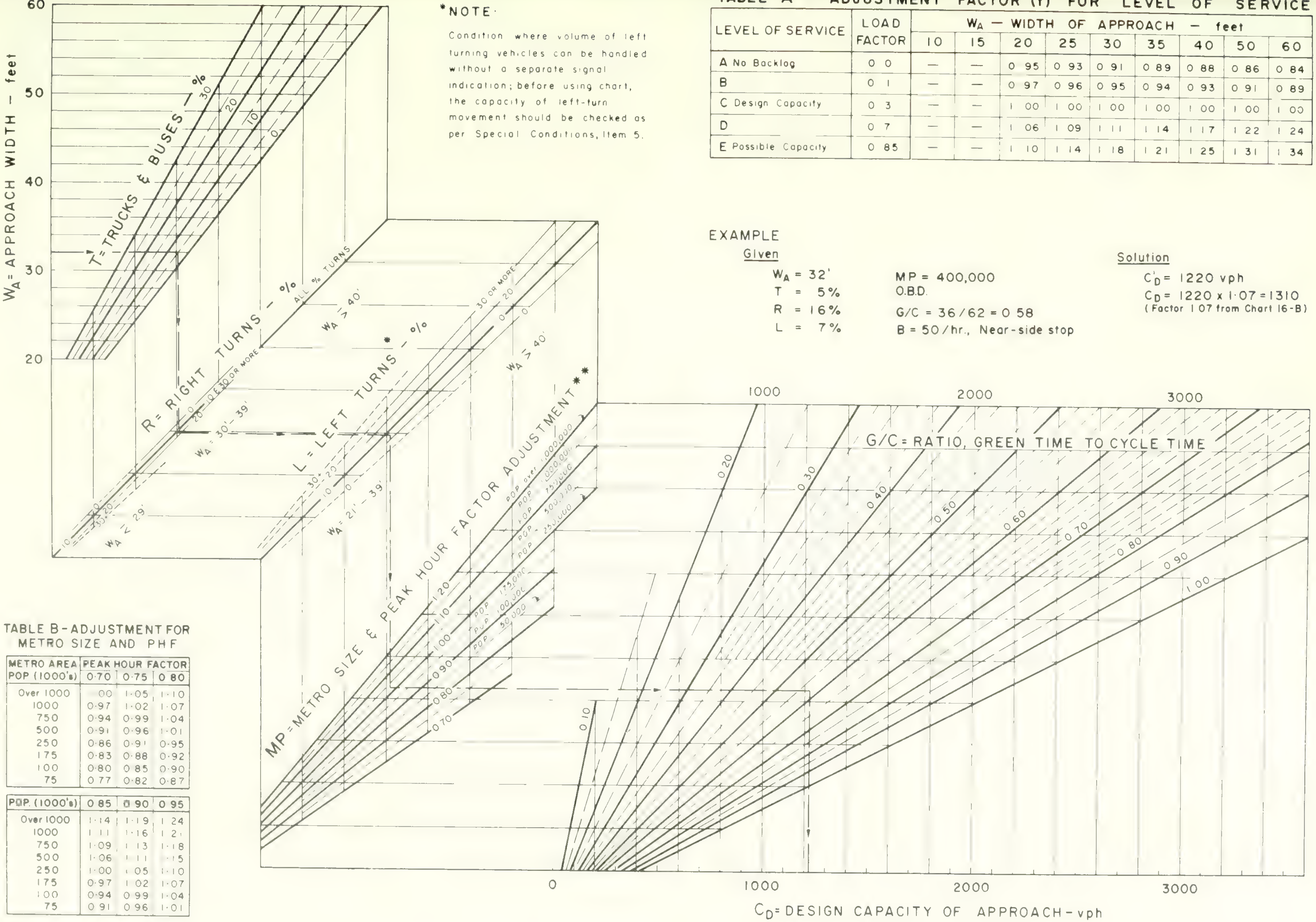
Then check in charts 17-A and 17-B the capacity of the left-turn lane with this signal timing. Entering chart A with $V_o = 350$ v.p.h. and proceeding to right and bottom with

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	0.95	0.93	0.91	0.89	0.88	0.86	0.84
B	0.1	—	—	0.97	0.96	0.95	0.94	0.93	0.91	0.89
C Design Capacity	0.3	—	—	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	1.06	1.09	1.11	1.14	1.17	1.22	1.24
E Possible Capacity	0.85	—	—	1.10	1.14	1.18	1.21	1.25	1.31	1.34

*NOTE·

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.



EXAMPLE

Given

$W_A = 32'$
$T = 5\%$
$R = 16\%$
$L = 7\%$

MP = 400,000
O.B.D.

G/C = 36/62 = 0 58
B = 50/hr., Near-side stop

Solution

$C'_D = 1220$ vph
$C_D = 1220 \times 1{\cdot}07 = 1310$
(Factor 1 07 from Chart 16-B)

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR		
	0·70	0·75	0 80
Over 1000	1·00	1·05	1·10
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·86	0·91	0·95
175	0·83	0·88	0·92
100	0·80	0·85	0·90
75	0·77	0·82	0·87

POP. (1000's)	0 85	0 90	0 95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·13	1·18
500	1·06	1·11	1·15
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·96	1·01

* * Use Table B if PHF is known to find adjust factor, otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH PARKING
O.B.D., FRINGE & RESIDENTIAL AREA
CHART 6

6
FRNG.
O.B.D.
RES.

$T_o=20/350=6\%$, $G/C=0.57$ and $T_3=30/240=13\%$ (see arrows), the design capacity of the left-turn lane is $C_{D3}=270$ v.p.h. Chart 17-A governs here as the value of 80 v.p.h. for $C=72$ seconds from chart 17-B (see arrows) is much less. Thus, the indicated left-turn volume of 240 v.p.h. can be accommodated.

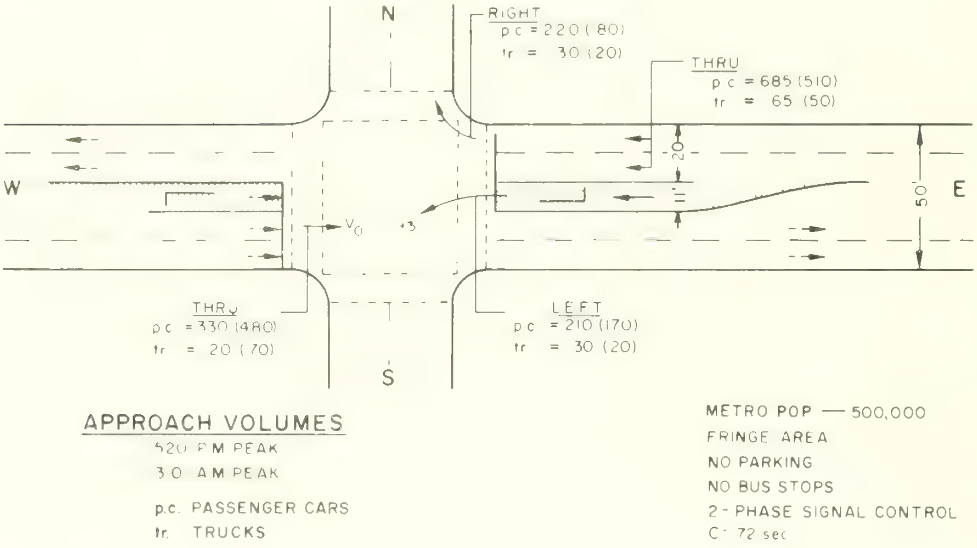


Figure 5.—Problems 15 and 20 illustrated.

The G/C required to handle the p.m. peak-hour traffic on the east approach without exceeding design capacity, therefore, is 0.57 (controlling value for the through-plus-right movement).

The required length of left-turn lane is obtained from chart 17-E. Using $V_3=240$ v.p.h., $C=72$ seconds and $T_3=13\%$, find $D_3=270$ feet (see arrows).

RIGHT-TURN LANE.—Charts 17-C and 17-D give the design capacity of an exclusively right-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication. The following additional terms are introduced in the charts:

D_2 Effective length of right-turn lane, in feet, for the storage of turning vehicles, exclusive of crosswalk and taper.

a Width of right turn lane, in feet.

V_2 Volume of traffic turning right on one approach, in vehicles per hour.

T_2 Trucks and buses turning right, expressed as a percentage of the total right-turn volume, V_2 on one approach.

C_{D2} Design capacity of the separate lane for right-turn movement in vehicles per hour.

The capacity of a right-turn lane is largely dependent on the proportion of truck traffic T_2 and the G/C ratio available for movement of traffic in the lane. Charts 17-C and 17-D show design capacity in terms of these factors. Right-turn lane capacity is also dependent on the radius of the turn, the amount of pedestrian interference, and the length of lane provided. From available data, distinction has been made for two general conditions in regard to radius and pedestrians. In each case, the intersection sketches shown with the charts are indicative of these conditions.

Chart 17-C represents average normal curb return, that is, corner radius at edge of pavement, and pedestrian interference, based on an average flow of 600 vehicles per hour of green including 5 percent trucks. The width of lane is not a factor here because either the small corner radius or pedestrian interference, or both, is the principal control. Chart 17-C is applicable generally to ordinary street conditions and geometry of streets in built-up areas.

Better conditions with adequate curb return and moderate to light pedestrian interference are represented in chart 17-D, based on a design capacity of right-turn lane of 800 vehicles per hour of green with 5% trucks per 10 feet of lane width. With no pedestrian interference, the design capacity base is 900 vehicles per hour of green with 5% trucks per 10 feet of lane width. Such high-type conditions are not covered directly in the *Manual;* however, where the movement is free of pedestrians, values corresponding to turning lanes with separate signal indication are suggested in the *Manual.* Accordingly, service volume bases of 800 and 900 v.p.h. are considered to be indicative of these designs.

The length of right-turn lane required to handle the demand volume of right-turning vehicles, or the volume corresponding to the capacity of the right-turn lane, is given in chart 17-E. Right-turn lane application, method of measurement, and the design of taper are the same as discussed for the left-turn lane above.

To determine the capacity of an intersection approach with separate right-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication, the following procedure should be followed:

(1) Obtain C_{D2} from chart 17-C or 17-D.

(2) Obtain C_D of combined through and left-turning movement from Charts 3–6 or 15, using:

W_A approach width exclusive of right-turn lane, and

$R=0\%$.

(3) Obtain D_2 from chart 17-E for V_2 or C_{D2}, whichever governs.

(4) Determine possible capacity, when required:

Right-turn lane, $C_{P2}=1.3\times C_{D2}$

Through and left-turn movement, combined, $C_P=f\cdot C_D$.

Values for f are given in table A on charts 3–6 or 5.1

In this procedure, the design capacity of the intersection approach cannot be taken as the sum of separate values of the capacity of the right-turn lane and that of the through-plus-left lanes, owing to the traffic pattern demand of the intersection. Separate comparisons of the volumes of individual movements with their capacities are necessary. This aspect has been previously covered with regard to separate left-turn lanes.

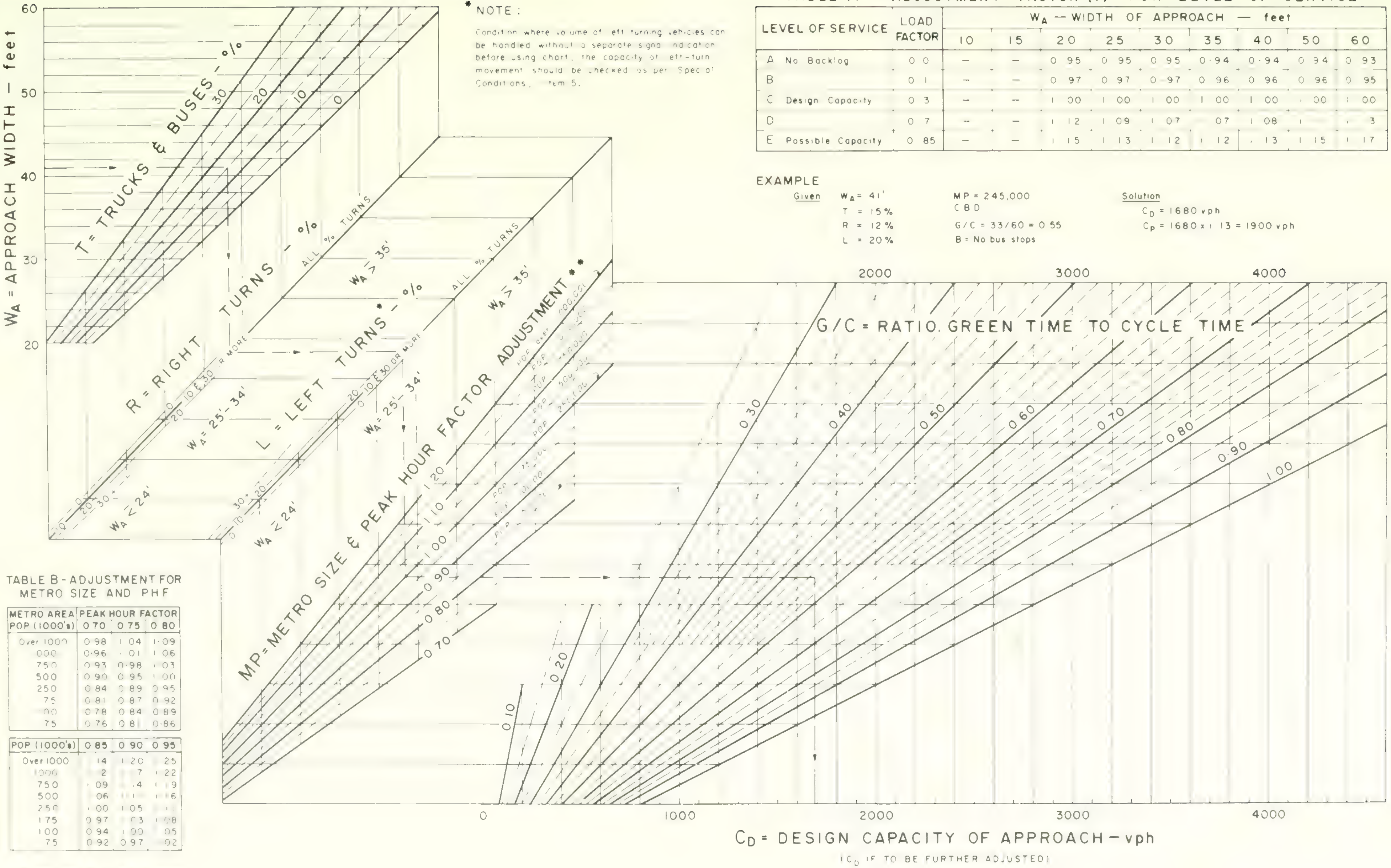
Problem 16

What is the maximum approach volume that can be accommodated without exceeding design capacity for the conditions indicated in figure 6. What maximum approach volume can be accommodated without exceeding possible capacity? What should be the length of the right-turn lane?

Solution: Using the same percentage of trucks in the right-turning movement as in the total approach volume, and $G/C=36/62=0.58$ in chart 17-C, find $C_{D2}=330$ v.p.h.

From chart 3, using $W_A=22$, $T=12\%$, $R=0\%$, $L=5\%$, $MP=250{,}000$ population, and $G/C=0.58$, the design capacity of combined through and left-turning movement is $C_D=870$ v.p.h. On this basis, the right-turning volume $V_2=(870\times 25)\div(100-25)=290$ v.p.h. Because this is less than C_{D2}, the maximum approach volume without exceeding design capacity is $870+290=1{,}160$ v.p.h.

Possible capacity of the through-plus-left movement is $870\times 1.20=1{,}040$ v.p.h., and corresponding right-turning volume is $(1{,}040\times 25)\div(100-25)=350$ v.p.h. The latter is less than the possible capacity of the right-turn lane ($C_{P2}=330\times 1.30=440$ v.p.h., from chart 17-C). The maximum approach volume without exceeding possible capacity is, therefore, $1{,}040+350=1390$ v.p.h. The length of right-turn lane required to handle the volume of 290 and 350 v.p.h. is found in chart 17-E; $D_2=280$ and 330 feet, respectively.



* NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	0.95	0.95	0.95	0.94	0.94	0.94	0.93
B	0.1	—	—	0.97	0.97	0.97	0.96	0.96	0.96	0.95
C Design Capacity	0.3	—	—	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	1.12	1.09	1.07	1.07	1.08	[illegible]	[illegible]
E Possible Capacity	0.85	—	—	1.15	1.13	1.12	1.12	1.13	1.15	1.17

EXAMPLE

Given
$W_A = 41'$
T = 15%
R = 12%
L = 20%

MP = 245,000
CBD
G/C = 33/60 = 0.55
B = No bus stops

Solution
$C_D = 1680$ vph
$C_P = 1680 \times 1.13 = 1900$ vph

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	0.98	1.04	1.09
1000	0.96	1.01	1.06
750	0.93	0.98	1.03
500	0.90	0.95	1.00
250	0.84	0.89	0.95
175	0.81	0.87	0.92
100	0.78	0.84	0.89
75	0.76	0.81	0.86

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.20	1.25
1000	1.12	1.17	1.22
750	1.09	1.14	1.19
500	1.06	[illegible]	1.16
250	1.00	1.05	[illegible]
175	0.97	1.03	1.08
100	0.94	1.00	1.05
75	0.92	0.97	1.02

** [illegible] Table B [illegible]

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET — NO PARKING — C.B.D.

CHART 7

7

C.B.D.

BOTH LEFT- AND RIGHT-TURN LANES.—For the condition where lanes are added for both left and right turns, chart 17 is used as before to find capacities and lengths of the separate turning lanes. The following steps are used in the analysis of the intersection approach:

(1) Obtain C_{D3} from charts 17-A or 17-B, and C_{D2} from charts 17-C or 17-D.

(2) Obtain C_D of through movement from charts 3–6, using

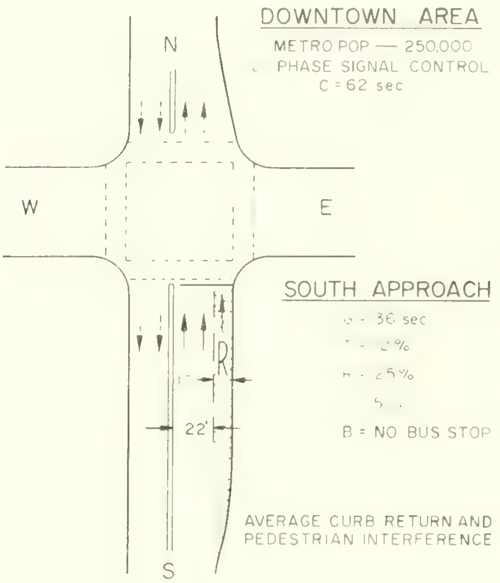


Figure 6.—Problem 16 illustrated.

W_1 = approach width exclusive of turning lanes, and

$L = 0\%$ and $R = 0\%$.

(3) Obtain D_2 and D_3 from chart 17-E.

(4) Determine possible capacity, when required:

Left-turn lane, $C_{P3} = 1.3 \times C_{D3}$.

Right-turn lane, $C_{P2} = 1.3 \times C_{D2}$.

Through movement, $C_P = f \times C_D$ (obtain values of f from table A on charts 3–6 or 15).

Problem 17

What is the maximum approach volume that can be handled on the east approach, shown in figure 7, without exceeding design capacity? A large lumber mill to the north on the crossroad accounts for a sizable proportion of vehicles and high percentage of trucks turning right.

Solution: Assuming a large through movement from the east, so that chart 17-B governs, the design capacity of the separate left-turn lane is $C_{D3} = 105$ v.p.h. The design capacity of the separate right-turn lane, using chart 17-*D* ($G/C = 31/56 = 0.55$, $a = 12$, $T_2 = 30\%$, and curve II), is $C_{D2} = 480 \times 0.90 = 430$ v.p.h. Factor of 0.90, according to the footnote in chart 17, is to account for the rural condition of design capacity based on level of service B.

The design capacity of the through movement is found in chart 15, using $W_1 = 22$, $T = 6\%$, $R = 0\%$, $L = 0\%$ and normal rural conditions; $C_D = 920$ v.p.h.

On this basis, $V_3 = (920 \times 7) \div (100 - 7 - 28) = 100$ v.p.h., and $V_2 = (920 \times 28) \div (100 - 7 - 28) = 395$ v.p.h.

Since V_3 and V_2 are less than C_{D3} and C_{D2}, the maximum approach volume which can be handled without exceeding design capacity is $920 + 100 + 395 = 1{,}415$ v.p.h.

Turning lanes with separate signal indication

Chart 18, which is similar to chart 17, gives the design capacity and the required length of a right- or left-turn lane, when traffic on this lane moves on a green indication that is separate from the green indications for other traffic on the approach—that is, a right- or left-arrow indication for the turning movement. The only additional term introduced in chart 18 is G', the green interval, in seconds, of separate signal indication for the movement of traffic on a separate turning lane.

With a separate signal indication, the right- or left-turning movement is assumed to be free from interference of other traffic streams and pedestrian movements. For intersections having normal street geometry, the design capacity of a turning lane, corresponding to level of service C, is 800 v.p.h. with 5% trucks in the stream per 10 feet of lane width. For high-type intersections with channelization, the design capacity of a turning lane, corresponding to a level of service C, is increased to 900 v.p.h. including 5% trucks per 10 feet of lane width. The base of 900 v.p.h. for high-type design represents an appropriately adjusted value of the 800 v.p.h. base for normal design indicated in the *Manual.*

Charts 18-A and 18-B give a solution for design capacity in terms of G'/C, width of lane a (10, 11, or 12 feet), and percentage of trucks and buses turning, T_2 or T_3. The design capacity of the turning lane for a given set of conditions is the same whether the movement is to the left or to the right and whether the lane is within the normal pavement width of the approach or is an added lane.

The possible capacity of a turning lane with a separate signal indication is 1.3 times the design capacity given in charts 18-A and 18-B. Values for other levels of service may be adjusted in accordance with factors given in table 5. For design capacity in rural areas, predicated on level of service B, the results in charts 18-A and 18-B usually would be multiplied by a factor of 0.90.

Chart 18-C provides a solution for the required length of turning lane that is identical with chart 17-E. Usually the separate signal phase is green while other through movements are stopped, and the storage lane must be long enough to prevent blocking of a through lane. To assure that all or nearly all vehicles are accommodated during each phase, the desirable length given in the chart, based on twice the average number of turning vehicles arriving per cycle, should be used where possible.

Another consideration in the determination of turning lane lengths with separate signal indication is that the lane should also be long enough to allow entry of turning vehicles past a line of stopped, through vehicles. To achieve this, the length of turning lane can also be determined in chart 18-C by using the through volume divided by the number of through lanes to enter the chart. The minimum length found, based on 1.5 times the average number of vehicles arriving per cycle, is the design indicated to meet the requirement of through traffic storage. Frequently, this aspect calls for a longer lane than may be required just to store the turning vehicles. In any event, the larger of the two determinations should be used as the length of turning lane.

The procedure for determining the maximum approach volume without exceeding capacity is the same as that previously explained in conjunction with chart 17. With a left-turn lane having a separate signal indi-

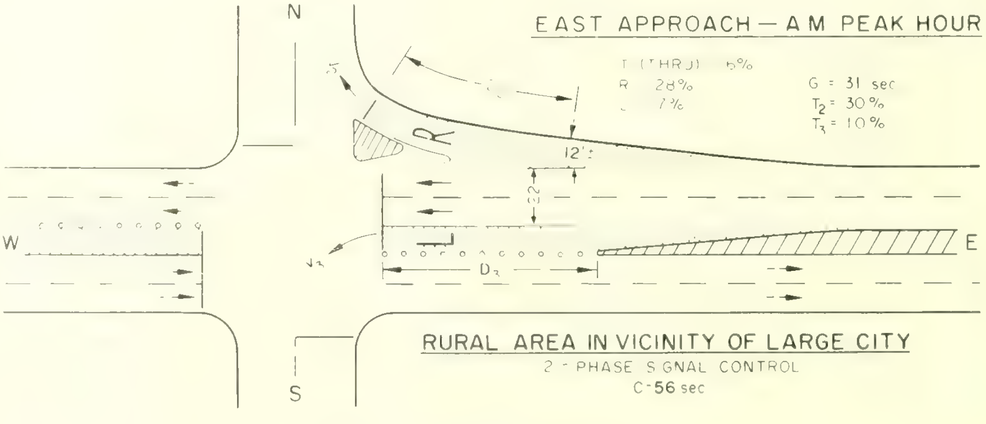


Figure 7.—Problem 17 illustrated.

*NOTE

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0.95	0.95	0.94	0.94	0.94	0.93
B	0·1	—	—	0·97	0·97	0.97	0.96	0.96	0.96	0.95
C Design Capacity	0.3	—	—	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	1.12	1.09	1.07	1.07	1.08	1.11	1.13
E Possible Capacity	0.85	—	—	1.15	1.13	1.12	1.12	1.13	1.15	1.17

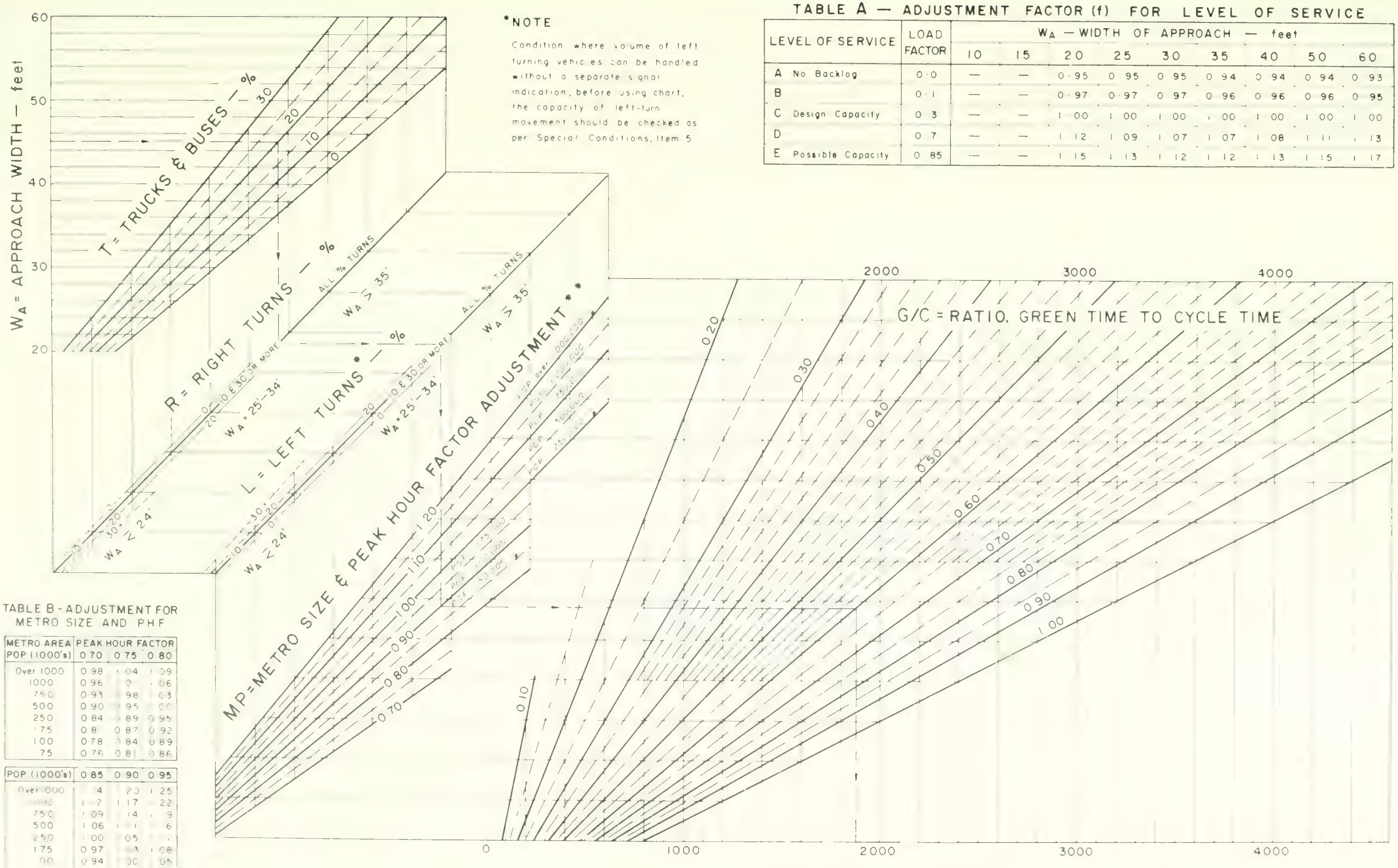


TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	0.98	1.04	1.09
1000	0.96	[illegible]	1.06
750	0·93	0.98	1.03
500	0.90	0.95	1.00
250	0.84	0.89	0.95
175	0.8	0.87	0.92
100	0·78	0.84	0.89
75	0.76	0.81	0.86

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.20	1.25
1000	1.12	1.17	1.22
750	1.09	1.14	1.19
500	1.06	1.11	1.16
250	1.00	1.05	[illegible]
175	0.97	1.03	1.08
100	0.94	1.00	1.05
75	0.92	0.97	1.02

** Use Table B if PHF is known [illegible] otherwise [illegible] directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — NO PARKING — FRINGE AREA & O.B.D.

CHART 8

8

FRNG.
O.B.D.

cation, the capacity of the through-plus-right lanes is found by using W_A exclusive of the left-turn lane and $L=0\%$ in charts 3–6 or 15. With a right-turn lane having a separate signal indication, the capacity of the through-plus-left lanes is found by using W_A exclusive of the right-turn lane and $R=0\%$ in charts 3–6 or 15.

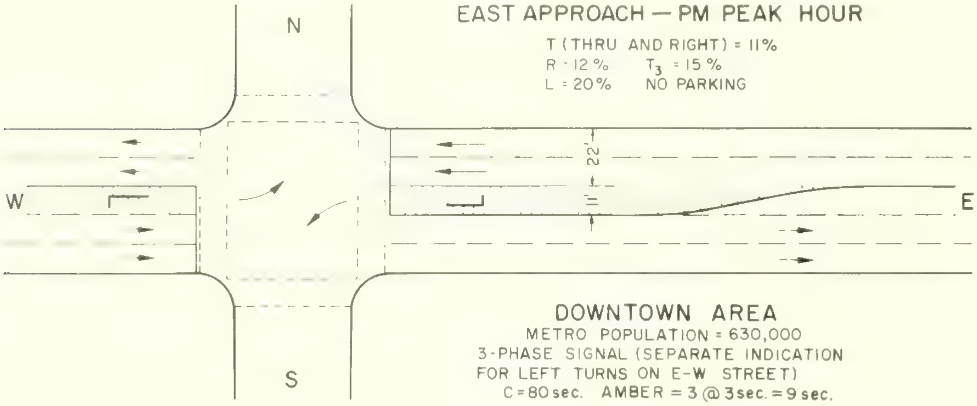

N
EAST APPROACH — PM PEAK HOUR
T (THRU AND RIGHT) = 11%
R = 12%
T3 = 15%
L = 20%
NO PARKING
W
22'
11'
E
DOWNTOWN AREA
METRO POPULATION = 630,000
3-PHASE SIGNAL (SEPARATE INDICATION FOR LEFT TURNS ON E-W STREET)
C = 80 sec. AMBER = 3 @ 3 sec. = 9 sec.
S

Figure 8.—Problem 19 illustrated.

Problems 18 and 19 demonstrate the use of chart 18.

Problem 18

Determine the design capacity and minimum length of a right-turn lane on a major street, 10 feet wide, normal conditions assumed, for which a separate signal indication of 25 seconds is used out of an 80-second cycle, and on which trucks comprise 6% of the total turning traffic. What would be the design capacity and length required for the same turning volume and signal timing if the turning lane is on an expressway at grade (high-type design)?

Solution: From chart 18-A, using $G'/C=25/80=0.31$, $a=10$, and $T_2=6\%$ (see arrows), find on the major street $C_{D2}=260$ v.p.h.; $D_2=220$ feet. From Chart 18-B, using $G'/C=0.31$, $a=12$, and $T_2=6\%$ (see arrows) find on the expressway $C_{D2}=350$ v.p.h.; $D_2=400$ feet.

Problem 19

What should be the green interval for each phase on the east approach of the intersection shown in figure 8 for operation at design capacity, if a total approach volume of 790 v.p.h. is to be accommodated? What will be the green interval for the movement of traffic on the cross street?

Solution: Volume of left-turning traffic is 20% of 790=160 v.p.h. The volume of through-plus-right movements to be accommodated on a width of 22 feet is $790-160=630$ v.p.h. The proportion of green time required for this movement, from chart 3, with $W_A=22$, $T=11\%$, $R=12\div(100-20)=15\%$, $L=0\%$, $MP=$population of 630,000 and $C_D=630$, is $G/C=0.40$. $G=80\times0.40=32$ seconds.

The proportion of green time required for the separate phase of left-turn lane is obtained from chart 18-A. Using a volume of 160 v.p.h., $T=15\%$, and $a=11$ feet, obtain $G'/C=0.19$. Hence $G'=80\times0.19=15$ seconds. The length of left-turning lane required, from chart 18-C, is $D_3=200$ feet. Green time available for movement of traffic on the cross street is $80-32-15-9$ (for amber) $=24$ seconds.

Problem 20

Determine the total length of green indication required during the a.m. peak to accommodate the traffic demand at design capacity on the east approach of the intersection shown in figure 5.

Solution: Signal timing based on a combined through-plus-right volume is obtained from chart 4 using $W_A=20$, $T=(20+50)\div(200+560)=9\%$, $R=200/760=26\%$, $L=0\%$, $MP=$population of 500,000, and approach volume$=200+560=760$ v.p.h.; find $G/C=0.43$.

Check in charts 17-A and 17-B whether the left-turning volume can be handled without a separate signal indication. Using in chart 17-A $V_o=550$, $T_o=70/550=13\%$, $G/C=0.43$ and $T_3=20/190=11\%$, C_{D3} is found to be negligible; thus, chart 17-B governs and shows a design capacity of 80 v.p.h. The 190 v.p.h turning left, therefore, must be handled either with a much larger G/C on the east approach, or with a separate signal indication for the left turn—three-phase control.

Re-entering chart 17-A with $V_o=550$, $T_o=13\%$ at the upper left, and with $C_{D3}=V_3=190$ v.p.h. and $T_3=11\%$ at the bottom, find the intersection of $G/C=0.70$. Considering a separate phase for the left-turning movement, $G'/C=0.23$ is indicated in Chart 18-A, using $C_{D3}=V_3=190$ v.p.h., $T_3=11\%$, and $a=11$ feet.

Thus, without a separate signal indication for the left turn, 2-phase control, the G/C required for the east approach is 0.70. With a separate signal indication for the left turn, 3-phase control, the G/C required for the east approach (total) is equivalent to $0.43+(3/72=0.04$ amber$)+0.23=0.70$. The total G/C for the east approach is the same with either phasing. However, the west approach and the approaches on the north-south street should be analyzed before determining the phasing. Another alternative for handling traffic on the east approach may involve an advance green, with possibly a lesser total G/C. See *Part 3, Special Conditions, Left-Turn Lane on Advance Green Signal.*

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	0.95	0.95	0.95	0.94	0.94	0.94	0.93
B	0.1	—	—	0.97	0.97	0.97	0.96	0.96	0.96	0.95
C Design Capacity	0.3	—	—	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	1.12	1.09	1.07	1.07	1.08	1.11	1.13
E Possible Capacity	0.85	—	—	1.15	1.13	1.12	1.12	1.13	1.15	1.17

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

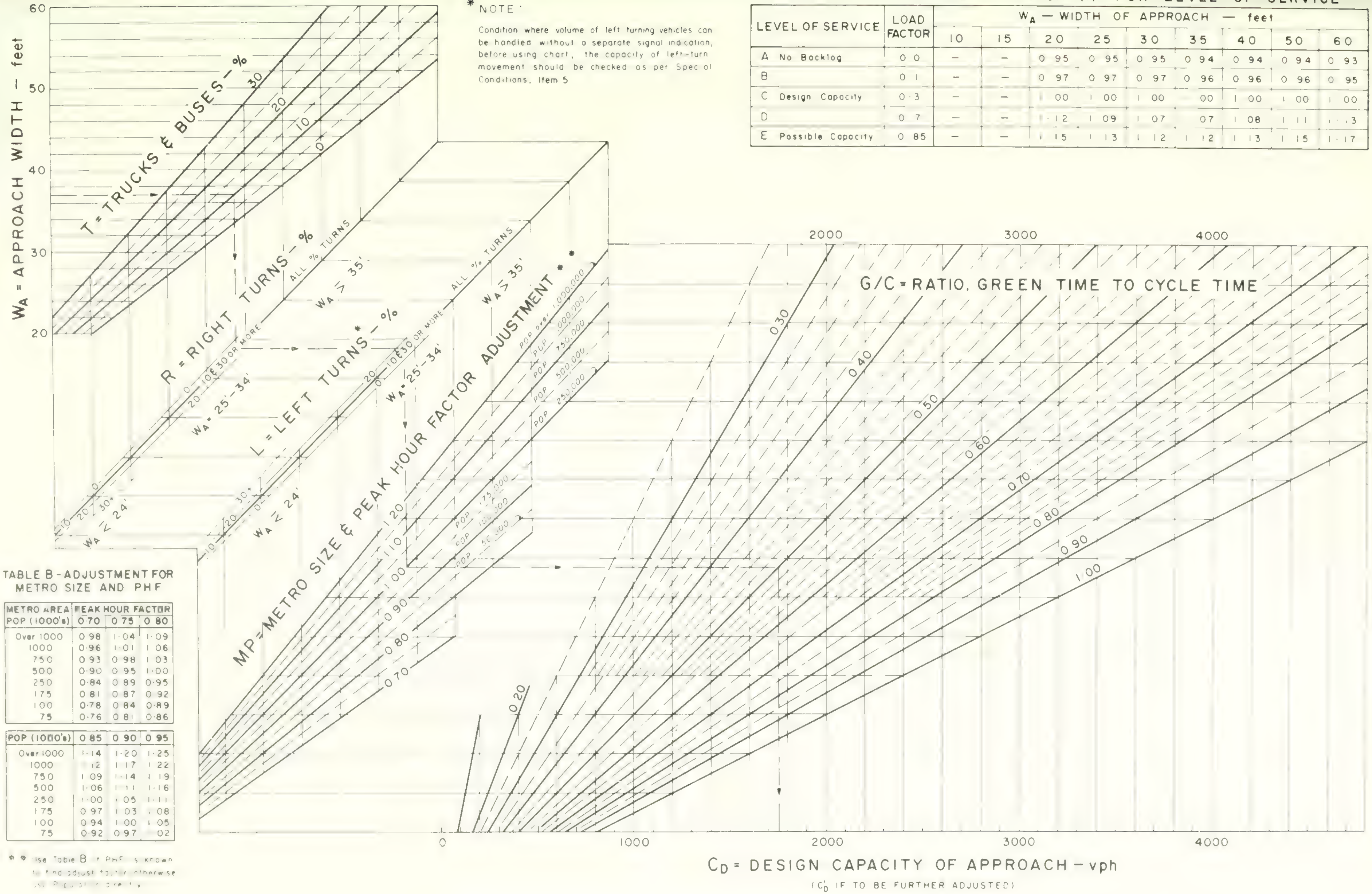


TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR		
	0.70	0.75	0.80
Over 1000	0.98	1.04	1.09
1000	0.96	1.01	1.06
750	0.93	0.98	1.03
500	0.90	0.95	1.00
250	0.84	0.89	0.95
175	0.81	0.87	0.92
100	0.78	0.84	0.89
75	0.76	0.81	0.86

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.20	1.25
1000	1.12	1.17	1.22
750	1.09	1.14	1.19
500	1.06	1.11	1.16
250	1.00	1.05	1.11
175	0.97	1.03	1.08
100	0.94	1.00	1.05
75	0.92	0.97	1.02

** Use Table B if PHF is known to find adjust factor otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — NO PARKING — RESIDENTIAL AREA

CHART 9

9

RES.

PART 2—1-WAY FACILITIES

Intersections With Average Conditions

As with two-way facilities charts were developed for evaluation of capacities of 1-way streets. For purposes of planning and preliminary design, chart 2 is presented in simplified form, using average conditions. It allows the determination of design capacity of 1-way street approaches with and without parking for various locations within metropolitan areas of different sizes.

Problem 21

An intersection on a 1-way street, 42 feet wide, with parking on both sides, located in the outlying business district of a metro area having a population of 750,000, is assumed to be operating under average conditions. What is the design capacity if the signal is so timed that $G/C=0.60$? What will be the possible capacity?

Solution: In chart 2, using $W_A=42$, *OBD–PKG BOTH SIDES* curve, and $G/C=0.60$, read for metro size of 750,000, $C_D=1{,}700$ v.p.h. From Table 2, $f=1.28$ and $C_P=1{,}700\times 1.28=2{,}170$ v.p.h.

Streets Without Parking

Charts 7, 8 and 9 include the adjustments for specific intersection conditions on 1-way streets without parking. Chart 7 is applicable to central business districts, chart 8 to fringe areas and outlying business districts, and chart 9 to residential areas.

Chart format and procedural steps in the solution of problems are similar to those for 2-way streets, including tables A and B, which provide adjustments for various levels of service and peak-hour factors. Where bus stops are present, chart 16-A is applicable.

Problem 22

A 41-foot street in a downtown area of a metro area of 245,000 population is converted to 1-way operation with no parking. Other conditions are $T=15\%$, $R=12\%$, $L=20\%$, $B=$ no bus stop, and 35% of the cycle time must be devoted to the cross street. What is the design capacity of the 1-way street if 2-phase signal control is used with $C=60$ seconds and each amber interval $=3$ seconds? What will be the possible capacity?

Solution: Green time that must be allotted to the cross street is 35% of $60=21$ seconds. Green time available for the 1-way street is $60-21-(2\times 3)=33$ seconds, and $G/C=33/60=0.55$.

Using chart 7, with $W_A=41$, $T=15\%$, $R=12\%$, $L=20\%$, $MP=$ population of 245,000 and $G/C=0.55$, obtain $C_D=1{,}680$ v.p.h.

$C_P=1{,}680\times 1.13=1{,}900$ v.p.h.

Streets With Parking—One Side

Charts 10 and 11 show capacity relations for 1-way streets with parking on one side only. The same charts are applicable whether the parking is on the left or on the right side. The only difference to be accounted for is when bus stops are present. With parking on the right, chart 16-B for bus stops, with parking, should be used to adjust design capacity values in charts 10 and 11. With parking on the left, chart 16-A for bus stops, with no parking, should be used to supplement the results in charts 10 and 11.

Problem 23

Determine the design capacity of a 33-foot, 1-way street with parking on the right side at a cross street which requires an 18-second green interval within a 75-second cycle. The intersection is situated in the fringe area of a city having a metro population of 650,000. Other conditions are $T=12\%$, $R=20\%$, $L=10\%$, bus stop on the near side serving 48 buses during the peak hour, and two 3-second amber periods per cycle.

Solution. Enter chart 10 with $W_A=33$ feet and proceed through the chart in accordance with the conditions given, using $G/C=1.00-18/75-6/75=0.68$ (see arrows); read $C_D=1{,}270$ v.p.h. In chart 16-B, using 48 buses per hour, fringe area, and $W_A=33$ feet, find a local bus factor of 1.09. Design capacity is $C_D=1{,}270\times 1.09=1{,}380$ v.p.h.

Problem 24

If, in Problem 23, all conditions remain the same, except that parking is shifted from the right to the left side, what will be the design capacity?

Solution: C_D, as above, is 1,270 v.p.h. Chart 16-A, with no parking, however, is applicable and produces a local bus factor of 0.85. Design capacity is $C_D=1{,}270\times 0.85=1{,}080$ v.p.h.

Streets With Parking—Both Sides

Capacities of 1-way streets with parking on both sides are covered in chart 12 for downtown and fringe areas, chart 13 for outlying business districts, and 14 for residential areas.

Problem 25

Determine the design capacity and possible capacity of a 41-foot, 1-way street with parking on both sides in the *CBD* of a metro area with a population of 175,000. Other conditions are: $T=15\%$, $R=28\%$, $L=9\%$, $G/C=0.48$, and no bus stops.

Solution: Proceed through chart 12 in accordance with the given conditions (see arrows); find $C_D=900$ v.p.h.

$C_P=900\times 1.28=1{,}150$ v.p.h.

Intersections With Separate Turning Lanes

Separate turning lanes on 1-way streets have the same operational characteristics and capacities as comparable turning lanes on 2-way facilities. Capacities of exclusive left- and right-turning lanes without a separate signal indication on 1-way approaches may be found in charts 27-C and 17-D, and required lengths in chart 17-E. With separate signal indication, the relations in chart 18 are applicable. The procedures for evaluating service volumes and capacities of 1-way approaches with turning lanes are the same as for 2-way facilities. A left-turn lane with separate signal indication is not as advantageous on a 1-way approach as on a 2-way approach with opposing traffic. However, separate signal indications for turning lanes on 1-way approaches may be employed in some channelization arrangements or in conjunction with separate pedestrian phases.

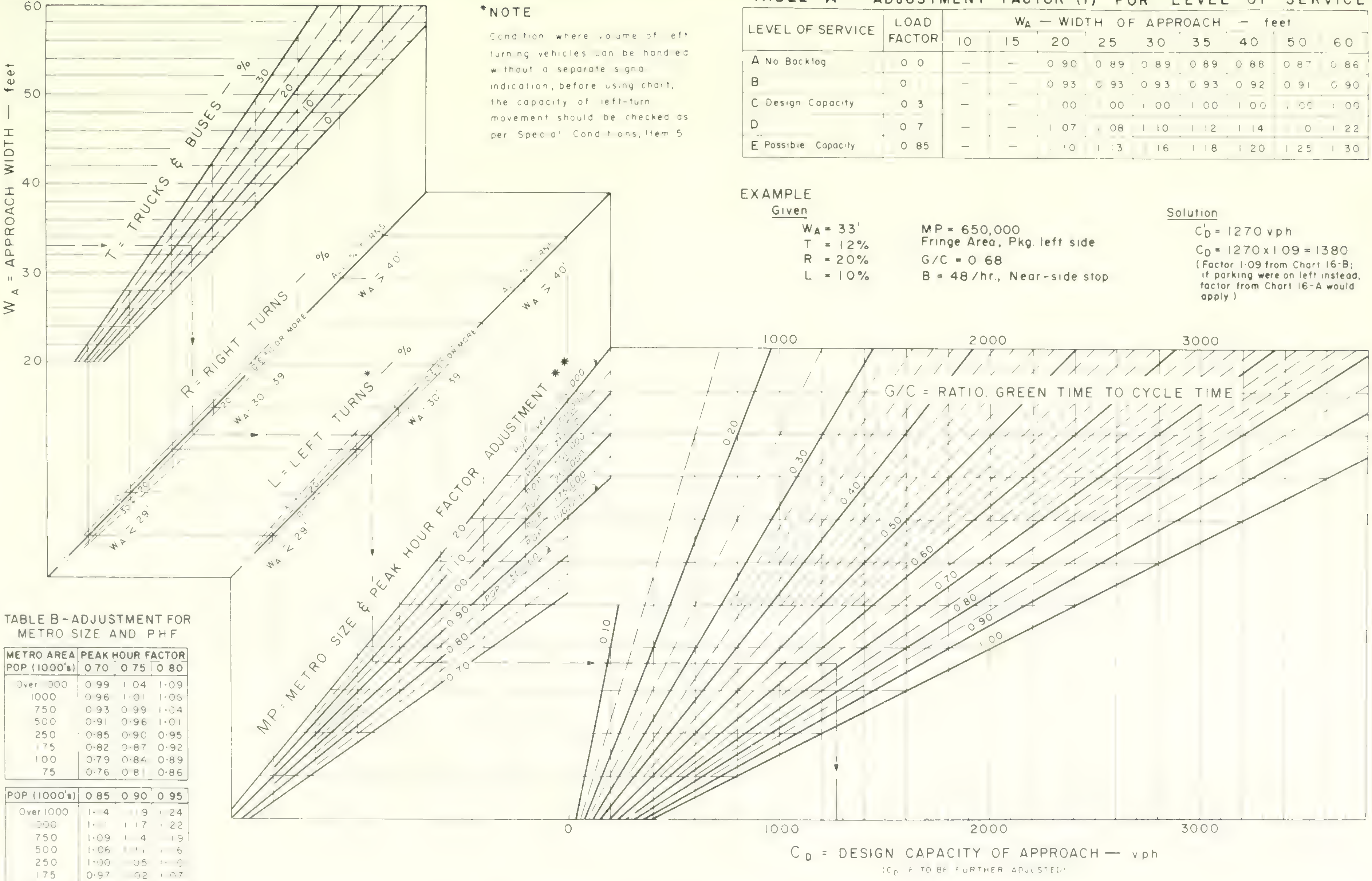
Problem 26

If all the conditions are the same as in problem 22, except that an 11-foot lane within the 41-foot street is designated exclusively for left turns, what is the design capacity of the approach?

Solution: For the through-plus-right movement from chart 7, using $W_A=41-11=30$ feet, $T=15\%$, $R=12\div(100-20)=15\%$, $L=0\%$, $MP=245{,}000$ pop., and $G/C=0.55$, $C_D=1{,}280$ v.p.h.

Left-turning volume $=(1{,}280\times 0.20)\div(100-20)=320$ v.p.h. Check for design capacity in chart 17-C, using $G/C=0.55$ and $T_L=15\%$, is $C_D=300$ v.p.h.; $C_P=300\times 1.3=390$ v.p.h.

If the left-turn lane is allowed to operate slightly under level of service C, operation equivalent to design capacity in the approach would be $1{,}280+320=1{,}600$ v.p.h. If no movement is to exceed design capacity, the approach volume would be $1{,}200+300=1{,}500$ v.p.h.



***NOTE**

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·90	0·89	0·89	0·89	0·88	0·87	0·86
B	0·1	—	—	0·93	0·93	0·93	0·93	0·92	0·91	0·90
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·07	1·08	1·10	1·12	1·14	[illegible]	1·22
E Possible Capacity	0·85	—	—	1·10	1·13	1·16	1·18	1·20	1·25	1·30

EXAMPLE

Given

W_A = 33'
T = 12%
R = 20%
L = 10%

MP = 650,000
Fringe Area, Pkg. left side
G/C = 0·68
B = 48/hr., Near-side stop

Solution

C'_D = 1270 vph
C_D = 1270 x 1·09 = 1380
(Factor 1·09 from Chart 16-B; if parking were on left instead, factor from Chart 16-A would apply.)

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·99	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·79	0·84	0·89
75	0·76	0·81	0·86

POP (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·97	1·02

** Use Table B if PHF is known [illegible]

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — PARKING ONE SIDE

C.B.D. & FRINGE AREA

CHART 10

10

C.B.D.
FRNG.

PART 3—SPECIAL CONDITIONS

CHARTS 1–15 cover the general conditions found at 2-way and 1-way intersections under traffic signal control. In addition, there are a number of other conditions in the analysis and design of intersections for which certain steps or instructions are to be followed in applying the charts.

Determination of Levels of Service Other Than at the Designated Design Capacity

Levels of service recommended in the *Manual* for design of intersections are C for urban conditions and B for rural conditions. Because of the complexity of intersection operation and the numerous adjustment factors required, some of which are empirical, there seems to be little justification for considering other levels of service in design. Accordingly charts 1–14 and 17–20 are predicated on a design capacity corresponding to level of service C. Chart 15, representing rural conditions, is based on level of service B. Possible capacity, or level of service E, is also considered in the analyses to establish the maximum attainable service volume. Pcssible capacity is determined by the application of conversion factors in table 2 to the chart design capacity values.

If desired, other levels of service can be evaluated by the application of factors listed in tables 3, 4 and 5, as follows:

(1) To find service volume for some other level of service for a given width of approach and other conditions: find design capacity on chart and multiply result by appropriate factor in table 3, 4 or 5.

(2) To find width of approach or G/C to accommodate a given traffic volume at a desired level of service: divide traffic volume by appropriate factor in table 3, 4 or 5 and enter chart to find the required element.

(3) To determine the level of service for a given traffic volume when all the intersection approach conditions and signal timing are known: find design capacity in chart; divide traffic volume by design capacity; compare result, V/C_D ratio, with factors in table 3, 4 or 5 to determine level of service. An illustration of this procedure may be found in problem 29.

Signal Systems Other Than Fixed Time

The capacity charts and analyses presented thus far are predicated on fixed-time signal control; that is, the signal setting remains unchanged during the analysis period. This also pertains to signal timing which may be varied during certain periods of the day—such as 3 dial control for which a different signal setting is used during the morning peak period, the off-peak periods, and the evening peak period. Utilization of the G/C ratio in capacity analyses is also applicable, with some modification, to actuated control and progressive systems.

Fully actuated control

The proportion of green time during a given hour, or the average G/C, for an intersection approach operated under fully actuated control is the sum of all the green intervals in seconds during the hour divided by 3,600. Because the green interval varies generally in accordance with demand, the actual proportion of green time within the hour cannot be predicted. However, the G/C ratio, based on the average green interval for the hour, would indicate the proportion of green time used. Therefore, since the signal timing varies approximately with the demand flow of vehicles per hour per lane on the several approaches, in accordance with relative capacities of each approach, there is a tendency for the average G/C ratios to approach the G/C ratios of fixed-time control. That is, the average G/C ratio for a given approach under actuated control is approximately equal to the ratio of the demand volume per lane on that approach to the sum of the demand volumes per lane on the two approaches controlling the signal timing during the hour, less the proportion of cycle used for amber periods. This, in essence, is the procedure used for capacity analyses of intersections operated under fixed-time control. Therefore, *for fully actuated signal control the same procedure as for fixed-time control may be employed as a close approximation of the G/C ratios for purposes of planning and design* of intersections. For improvement of existing intersections, field studies to determine average G/C ratios should be undertaken to achieve more precise results. Actually, the fully actuated control is more efficient than fixed-time control because the effect of full actuation is to more nearly fill or load each green interval. Accordingly, the method suggested for analyses in design provides a conservative approach.

Semi-actuated control

The estimation of the average G/C ratio requires consideration of the individual characteristics of the installation, including the green time setting on the minor approach and the minimum cycle length used. For intersection design, the following procedure is suggested:

(1) A trial average G/C for the critical approach on the minor road is determined, using an appropriate chart, as if the intersection were operated under fixed-time control.

(2) Usually, the green time on the minor road is kept to a minimum taking the hourly approach volume and peak-hour factor into consideration, but usually not less than a fixed interval of 15 seconds.

(3) A minimum C value is selected which would yield a G/C ratio approximately equal to or greater than that originally calculated.

(4) Generally, minimum C is selected to be not greater than 150 seconds, preferably 120 seconds or less. When there is no demand on the cross road, the green phase on the major facility continues and C is extended. However, when there are repeated calls from the side road, the signal alternates in accordance with the minimum C.

(5) The average G/C on the minor road is then adjusted to these controls and the design of the major road approaches is predicated on the remaining available G/C.

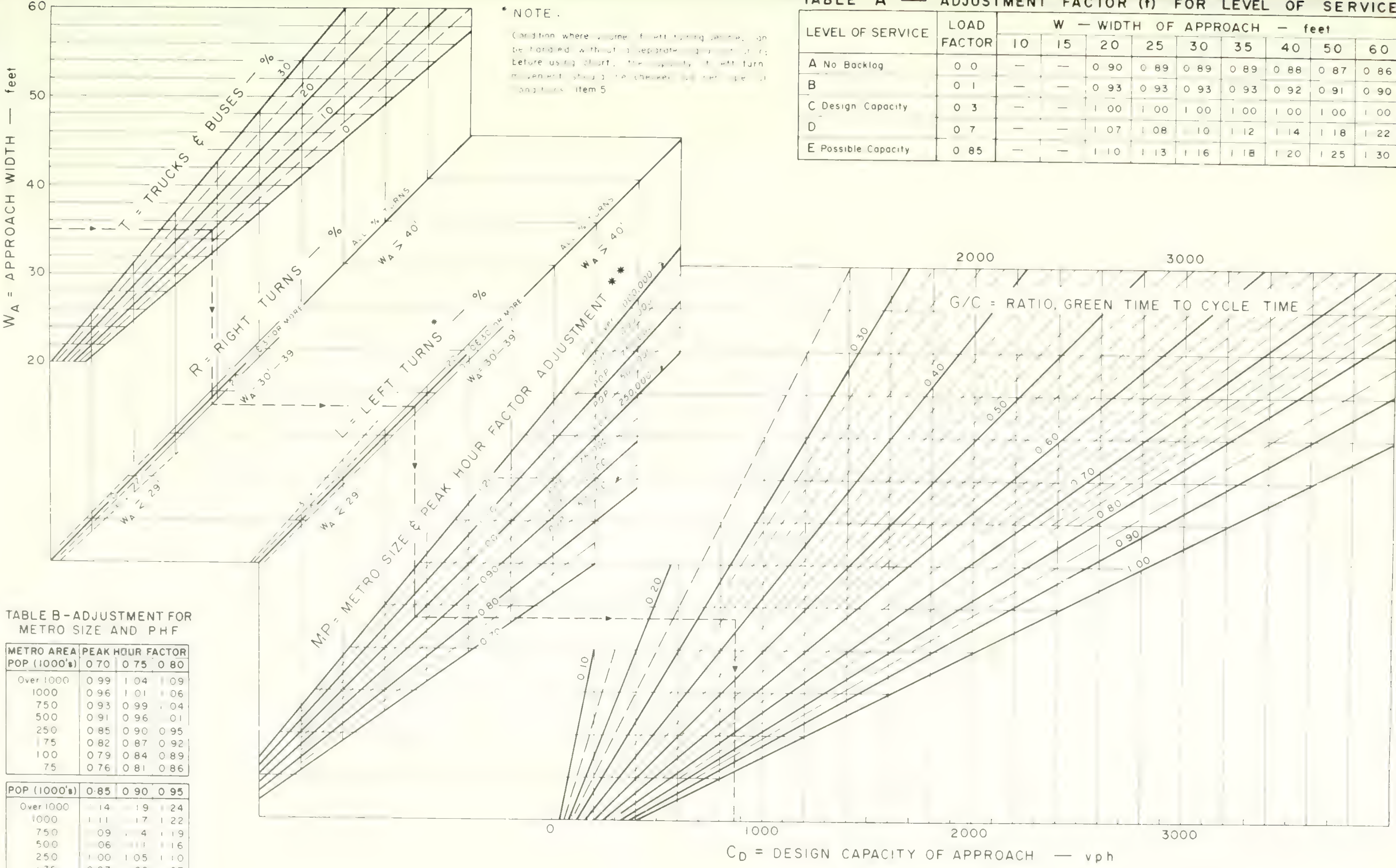
Local practice in the use of the length of the green interval on the minor road and the minimum cycle length may temper the values given in the above procedure. Caution should be exercised in determining the initial G/C in the first step of the procedure because semi-actuated control is used along major highways in conjunction with cross roads that carry light volumes and frequently accommodate peaks of short duration, such as 15 or 20 minutes. In such situations, an *hourly rate of flow* should be used as a basis for determining the G/C ratio and the G interval on the minor road. The procedure for analyzing approaches with high peaking characteristics is covered in this Part under the heading *Intersections with High Peaking Characteristics*.

Progressive signal systems

Signalizations that provide coordination between successive signals designed to keep platoons of traffic moving continuously through intersections, yield somewhat higher capacities than do isolated signals or series of signals without coordination. Stated another way, for the same volume and G/C, the level of service provided at an intersection within a progressive signal system would be somewhat higher than at an isolated, fixed-time signal controlled, intersection. The increase in service volume or capacity is difficult to estimate and may not be significant unless near perfect progression is achieved. To be safe, the increase usually is not taken into consideration in the design of geometric requirements of intersections. However, on 1-way streets with very efficient signal progression load factors equal to or approaching 1.00 might be achieved. The design and possible capacities under such circumstances, therefore, may be increased using tables 3 and 4, by the ratio of the f value for a load factor of 1.00 (extrapolated) to the f value for a load factor of 0.85. Usually, these values calculate to a reasonably consistent ratio of 1.05; thus, design and possible capacities for signalized intersections within an excellent progressive system may be determined by multiplying the chart values by 1.05.

Problem 27

A 2-way minor road intersects a major street in an outlying residential area of a city having a metro population of ½ million. The major facility requires a G/C of not less than



C_D = DESIGN CAPACITY OF APPROACH — vph

[illegible] BE FURTHER ADJUSTED

* NOTE.

Condition where volume of left turning vehicles can be handled without a separate [illegible]; before using chart, the capacity of left turn movement should be checked [illegible] [illegible] item 5

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	0.90	0.89	0.89	0.89	0.88	0.87	0.86
B	0.1	—	—	0.93	0.93	0.93	0.93	0.92	0.91	0.90
C Design Capacity	0.3	—	—	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	1.07	1.08	1.10	1.12	1.14	1.18	1.22
E Possible Capacity	0.85	—	—	1.10	1.13	1.16	1.18	1.20	1.25	1.30

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	0.99	1.04	1.09
1000	0.96	1.01	1.06
750	0.93	0.99	1.04
500	0.91	0.96	1.01
250	0.85	0.90	0.95
175	0.82	0.87	0.92
100	0.79	0.84	0.89
75	0.76	0.81	0.86

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.19	1.24
1000	1.11	1.17	1.22
750	1.09	1.14	1.19
500	1.06	1.11	1.16
250	1.00	1.05	1.10
175	0.97	1.02	1.07
100	0.94	0.99	1.04
75	0.91	0.97	1.02

** [illegible] B [illegible] otherwise [illegible]

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET — PARKING ONE SIDE
O.B.D. & RESIDENTIAL AREA
CHART 11

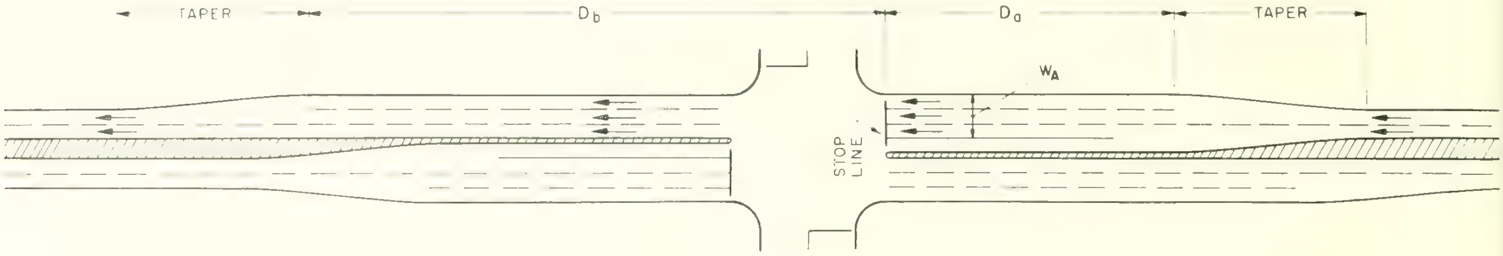
11

O.B.D.
RES.

0.75. The minor road has a demand volume of 200 v.p.h. in the predominant direction; other conditions on the approach are: $T=28\%$, R 20%, $L=35\%$, no parking, and no bus stops. A semi-actuated signal control is needed with a green interval preferably not less than 18 seconds on the crossroad. Determine the number of lanes on the minor approach and the signal timing for operation at design capacity.

Solution: Since the major facility is not to be altered, the number of lanes required on the minor approach may be estimated on the basis of a fixed-time signal. Because the average cycle is apt to be long, assume that only 5% of the cycle would be devoted to amber periods (A). The average G/C for the minor approach equals, tentatively, $1.00-0.75-0.05=0.20$. Using the indicated conditions in chart 4, $W_A=19$ feet, minimum. Employing 11-foot lanes for this facility, a W_A of 22 feet is selected. Using this width and other conditions, as before in chart 4, a G/C of 0.16 is required to handle 200 v.p.h.; or, holding $G/C=0.20$, a volume of 250 v.p.h. can be accommodated.

The minimum cycle length may be calculated as follows: G/C on major street, plus G on minor street$\div C$, plus A(per cycle)$\div C$ equals 1.00; for $G=18$ seconds, and $A=6$ seconds, $0.75+18/C+6/C=1.00$, or $C=95$ seconds. If a longer minimum cycle of, perhaps, 120 seconds is desired, G on the minor road can be increased to $120\times.20=24$ seconds; or, if 20 seconds of green is set with a 120-second cycle, $G/C=20/120=0.17$; then, additional green time can be devoted to the major street, so that $G/C=(1.00-0.17-0.05)=0.78$.



LENGTH OF WIDENING BEYOND INTERSECTION

LENGTH REQUIRED FOR: *			TAPER feet
ACCELERATION		MERGING	
DESIGN SPEED mph	D_b — feet		
40	200	D_b = 12 x G (G Green interval n seconds)	200
50	525		250
60	900		300

* Use the larger of two values but not less than 300 feet

LENGTH OF WIDENING IN ADVANCE OF INTERSECTION

LENGTH REQUIRED FOR: †			TAPER feet
DECELERATION		STORAGE	
DESIGN SPEED mph	D_a — feet	• Divide approach volume by number of lanes in W_A • Use volume per lane in chart 18-C, find $D_2 = D_a$ on desirable scale (minimum scale for restricted conditions)	
40	150		175
50	200		225
60	250		275

† Use the larger of two values but not less than 200 feet

Figure 9.—Intersection with widened approaches, length requirements.

Intersections With High Peaking Characteristics

Intersection capacities vary with city size and peaking characteristics. Normally, peak-hour factors are within a range of 0.80 to 0.90. Where such factors are not available, which often occurs in design, the capacity analysis may proceed with the knowledge only of metro population as provided for in charts 3–14.

Some intersection approaches serve localized traffic generators that produce high peaks within a 15- or 30-minute period, while the other approaches at the same intersection may be operating under normal peaking conditions. The low peak-hour factor on one approach under such circumstances should be accounted for in evaluation of capacities. This may be done, if the peak-hour factor is 0.70 or more, by using an appropriate adjustment factor in charts 3–14, found in table B on the charts for the particular metro size and PHF. Where the PHF is less than 0.70 in urban areas, the following procedure should be used:

(1) Find C_D in charts 3–14, using metro size directly.

(2) Divide C_D by the following average peak-hour factors: 0.80 for metro population of 50,000 or less, 0.85 for metro population of 100,000–750,000 and 0.90 for metro population of 1,000,000 or more.

(3) Multiply above by PHF determined for the approach to establish design capacity.

In rural areas, find C_D in chart 15 using $PHF=1.00$; then, multiply result by PHF relevant to the approach to determine design capacity.

Problem 28

In the outlying district of a metro area of one million population, a road intersecting an arterial facility serves an industrial plant. A G/C of 0.38 is available for moving traffic in the p.m. peak on the crossroad. During this period the volume in one direction on the critical approach is 840 v.p.h.; and, during the highest 15 minutes of the hour, there is a flow of 390 vehicles as determined by field measurements. Other conditions are: $T=5\%$, $R=38\%$, $L=7\%$, no parking, and no bus stops. Determine the number of lanes required on the crossroad approach.

Solution: Because of the high peak within the hour, a PHF of $840\div(4\times390)=0.54$ should be used in the analysis. To find the required width, the chart has to be entered with the demand volume. The demand volume, however, must be adjusted for the special condition of low peak-hour factor. The adjusted volume, as previously explained, but reversed for this problem, is $840\times0.90\div0.54=1{,}400$ v.p.h. Proceed through chart 4 from bottom to upper left using: $C_D=1{,}400$, $G/C=0.38$, $MP=$population of 1,000,000, $L=7\%$, $R=30\%$ or more, and find $W_A=36.5$ feet. Provide three 12-foot lanes, or $W_A=36$ feet.

Widened Approaches

Intersection capacities can be significantly increased by widening the traveled ways through the intersection. This may be accomplished in conjunction with one or more approaches by adding a traffic lane for a certain distance in advance of and beyond the crossroad; or, on streets with parking, the additional lane through the intersection can

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C.B.D.
FRNG.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	—	0·88	0·85	0·84	0·83	0·83	0·82
B	0·1	—	—	—	0·91	0·90	0·90	0·89	0·88	0·88
C Design Capacity	0·3	—	—	—	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	—	1·17	1·17	1·17	1·18	1·22	1·25
E Possible Capacity	0·85	—	—	—	1·25	1·25	1·25	1·27	1·32	1·37

EXAMPLE

Given

W_A = 41'
T = 15%
R = 28%
L = 9%

MP = 175,000
CBD
G/C = 0.48
B = No bus stops

Solution

C_D = 900 vph
C_P = 900 x 1.28 = 1150 vph

*NOTE

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

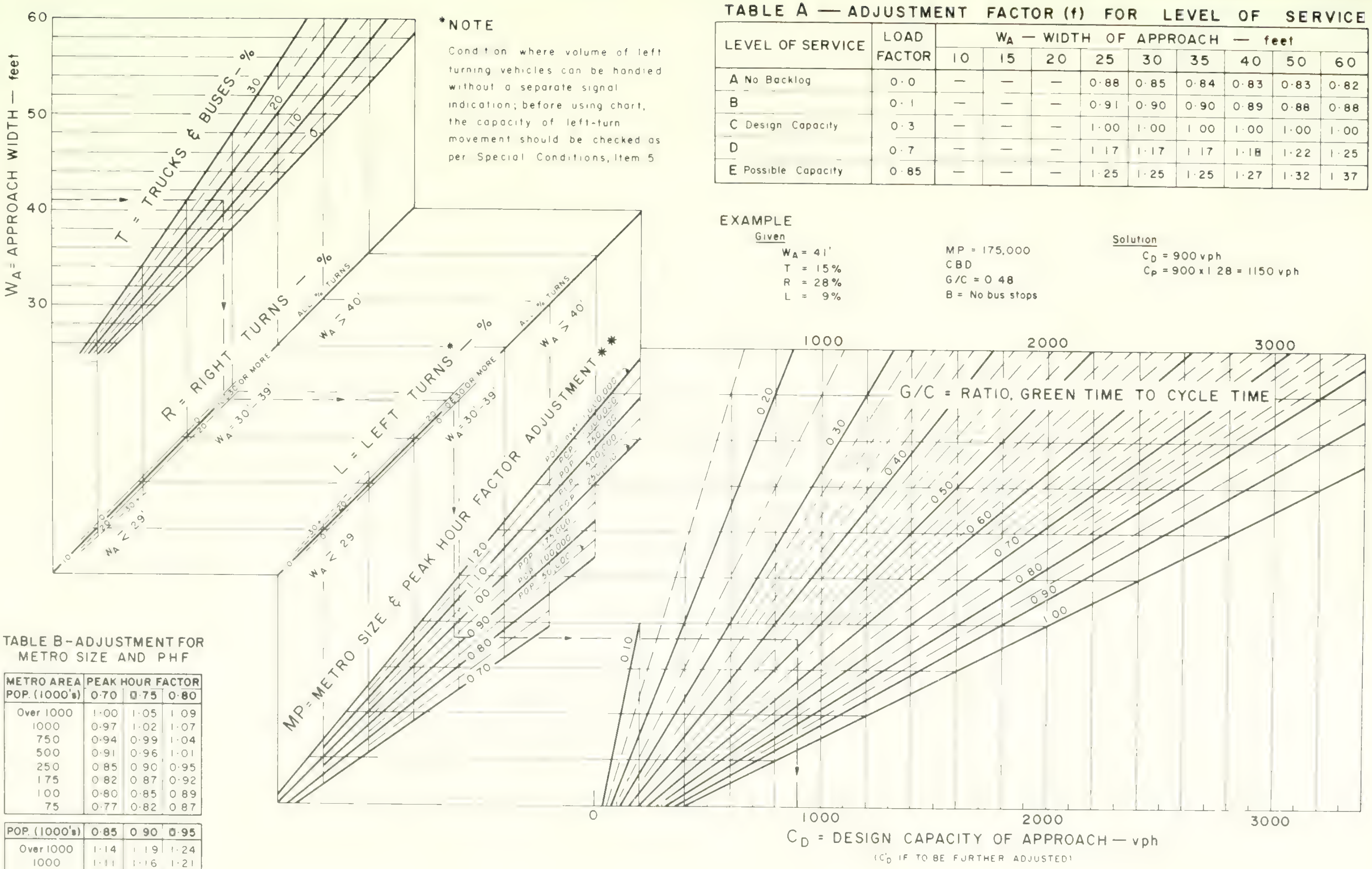

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR		
	0·70	0·75	0·80
Over 1000	1·00	1·05	1·09
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·80	0·85	0·89
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·14	1·18
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·92	0·96	1·01

** Use Table B if PHF is known to find adjust. factor, otherwise use [illegible] directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET — PARKING BOTH SIDES
C.B.D. & FRINGE AREA
CHART 12

be supplied by partial elimination of parking. Where the extra lane introduced is of sufficient length preceding and following the intersection, the service volumes and capacities will correspond to those for the total width of approach as if the added lane were continuous indefinitely.

Increased width through intersection—no parking

When the traveled way is widened for a sufficient distance in advance of the intersection and the same widening is continued for a sufficient distance beyond the intersection, enter charts 3–4, 7–9, or 15 with W_A equal to the total width of one approach including the widening on that approach, then use the chart in the normal manner. Required minimum lengths of widening are presented in figure 9. The larger of the two values indicated in the figure for the approach and departure should be used, but not less than 200 and 300 feet, respectively. The widening thus determined would be preceded and followed by appropriate tapers in accordance with design speed.

Elimination of parking through intersection

Where parking on a street approach is eliminated in advance of the intersection and beyond the intersection for a sufficient distance; use chart 3–4 or 7–9, instead of chart 5–6 or 10–14, as if there were no parking on the street. Required minimum distances for elimination of parking to achieve this condition are: First, elimination of parking in advance of the crossroad, measured back from the stop line on the approach, should be for a distance in feet equal to or greater than 8 times the green interval in seconds ($8G$), but not less than 250 feet; and second, elimination of parking beyond the crossroad, measured forward from the stop line on the approach, should be for a distance in feet equal to or greater than 12 times the green interval in seconds ($12G$), but not less than 350 feet.

Problem 29

An isolated intersection at a crossroad in a rural area with normal peaking characteristics is causing congestion on a 4-lane, divided highway. The 2-phase signal control is timed in balance with the demands and capacities of the intersecting facilities and allocates to the major highway a G/C of 0.65 within a 74-second cycle. All lanes are 12 feet wide, and separate left-turn lanes are provided in the median. Design speed is 50 m.p.h. Other conditions on the critical approach are: $T=8\%$, $R=10\%$, $L=5\%$. Determine the level of service on the one approach of the divided highway during the peak hour when the total approach volume is 1,400 v.p.h. Determine to what degree congestion may be alleviated by widening the highway through the intersection, as shown in figure 9, but using the same signal timing. If the uninterrupted flow design capacity of the highway at level of service B is 1,850 v.p.h., how would the signal timing be adjusted (assuming that the crossroad can be improved to operate on a lesser green-time allocation with a G of not less than 18 seconds) to bring the intersection design capacity in balance with the remainder of the divided highway?

Solution: Because the left-turning movement is accommodated in a separate lane, the through-plus-right movement is 95% of 1,400, or 1,330 v.p.h., and R with the left-turning movement removed is $10\div0.95=11\%$. In chart 15, using $W_A=24$, $T=8\%$, $R=11\%$, $L=0\%$ and $G/C=0.65$, find $C_D=1{,}100$ v.p.h. Ratio of approach volume to design capacity is $V/C_D=1{,}330\div1{,}100=1.21$. In table A of chart 15, f for level of service C is 1.11, and for level D is 1.23. Operation, therefore, is at level of service D.

By widening the approach through the intersection by one lane, C_D is increased to 1,530 v.p.h., using $W_A=36$ feet and no change in other conditions. Ratio of approach volume to design capacity is $V/C_D=1{,}330\div1{,}530=0.87$, which according to table A in chart 15, operation would be at level of service A. To achieve this design capacity, the lengths of added lane in advance of and beyond the intersection, in each case measured from the stop line on the approach, are as follows in accordance with figure 9:

D_a—for deceleration, length is 200 feet; for storage, using a volume of $1{,}330\div3=440$ v.p.h. in chart 18–C, length is 480 feet; D_a required is 480 feet plus taper of 225 feet.

D_b—for acceleration, length is 525 feet; for merging, length is $12\times0.65\times74=570$ feet; D_b required is 570 feet plus taper of 250 feet.

To bring the design capacity of the widened intersection in balance with the uninterrupted-flow design capacity of the highway approach, 1,850 v.p.h., the signal timing must be adjusted to accommodate a through-plus-right volume of $1{,}850\times0.95=1{,}760$ v.p.h. Using chart 15 with the other conditions as before, find $G/C=0.74$. To maintain a green interval of not less than 18 seconds on the crossroad (previously available), assuming an 8-second amber per cycle, the new cycle length is determined from the relation $0.74+18/C+8/C=1.00$; $C=100$ seconds. The lengths of widening to maintain volumes equivalent to design capacity become $D_a=870$ feet (or minimum of 650 feet), and $D_b=890$ feet.

Check for Capacity of Left Turn

Any intersection approach on a 2-way street that does not involve a separate left-turn lane should be checked for capacity of the left-turn movement. This may be done in the same manner as for a separate left-turn lane as the number of left-turning vehicles that can be accommodated with 2-phase control, whether on a separate lane or not, is governed either by the volume of traffic opposing the left turn or by the length of cycle. Charts 17-A and 17-B should be used to check the capacity of the left-turn movement. Such a check is necessary for every intersection involving 2-way streets. If the volume of left-turning vehicles exceeds the possible capacity as determined in charts 17-A and 17-B, serious congestion may result, the overall capacity of the approach may be materially reduced, and the results determined in charts 3–15 would be erroneous. When this is encountered, the left-turnin movement should be prohibited or, if feasibl accommodated on a separate signal indicatio

Problem 30

Check whether the left-turn volume i problem 6 can be handled satisfactorily.

Solution: Left-turn volume is 5% of 1,20 or 60 v.p.h. For the conditions given, it i found in chart 17-B that upwards of 80 v.p.h can be accommodated at design capacity therefore, the solution in problem 6 i satisfactory.

Problem 31

A 23-foot approach on a 2-way stree without parking in the central business district of a metro area of ½ million populatio operates under the following conditions approach volume 950 v.p.h., of which 13 v.p.h. turn right and 210 v.p.h. turn left opposing through volume during the sam period is 670 v.p.h.; trucks are generally 8% on all movements; a 36-second green within 2-phase, 60-second cycle is allotted to thi street; there are no bus stops. Determine th level of service on the approach

Solution: A left-turn check must always b made as a first step in the problem solution o a 2-way facility controlled by a 2-phase signal In chart 17–A using $V_o=670$, $T_o=8\%$ $G/C=0.60$ and $T=8\%$, it is found that few left-turning vehicles can be handled durin the given phase. Chart 17–B, therefore, is use and indicates that 96 v.p.h. turning left ca be accommodated at design capacity and $96\times1.3=125$ v.p.h. at possible capacity.

Because the left-turning volume of 21 v.p.h. significantly exceeds the capacity, the left lane on the approach may be largely nullified as a carrier of straight-through traffic during the peak hour. If this condition were ignored in the analysis, an erroneous answer from chart 3 would indicate 860 v.p.h. as the design capacity of the approach—using $W_A=23$, $T=8\%$, $R=14\%$, $L=22\%$, $MP=$ population of 500,000 and $G/C=0.60$—and table A on the chart would indicate that, with a factor of $950/860=1.11$, operation is at level of service D. Under these circumstances ther is no way of determining the level of service at which the intersection approach would b operating. For example, if the left lane was occupied by vehicles waiting to turn left throughout the hour, capacity would be equivalent to just one lane or C_D of about 600 v.p.h. and C_P of about 720 v.p.h. The latter is approximately equal to the through-plus-right volume of $950-210=740$ v.p.h. Thus, the through-plus-right movement would operate at approximately the possible capacity and the left-turning movement would exceed possible capacity or would operate at level of service F. A solution to this problem would entail either a prohibition of the left turn or an approach widened to include an exclusive left-turn lane using a separate signal phase.

Non-Deterring Turning Movements

On the intersection approach of a 2-way facility where the right-turning path is reasonably direct and pedestrian interference is minor, the right-turning movement may be

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O.B.D.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet 10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	—	0.88	0.85	0.84	0.83	0.83	0.82
B	0.1	—	—	—	0.91	0.90	0.90	0.89	0.88	0.88
C Design Capacity	0.3	—	—	—	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	—	1.17	1.17	1.17	1.18	1.22	1.25
E Possible Capacity	0.85	—	—	—	1.25	1.25	1.25	1.27	1.32	1.37

* NOTE:
Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

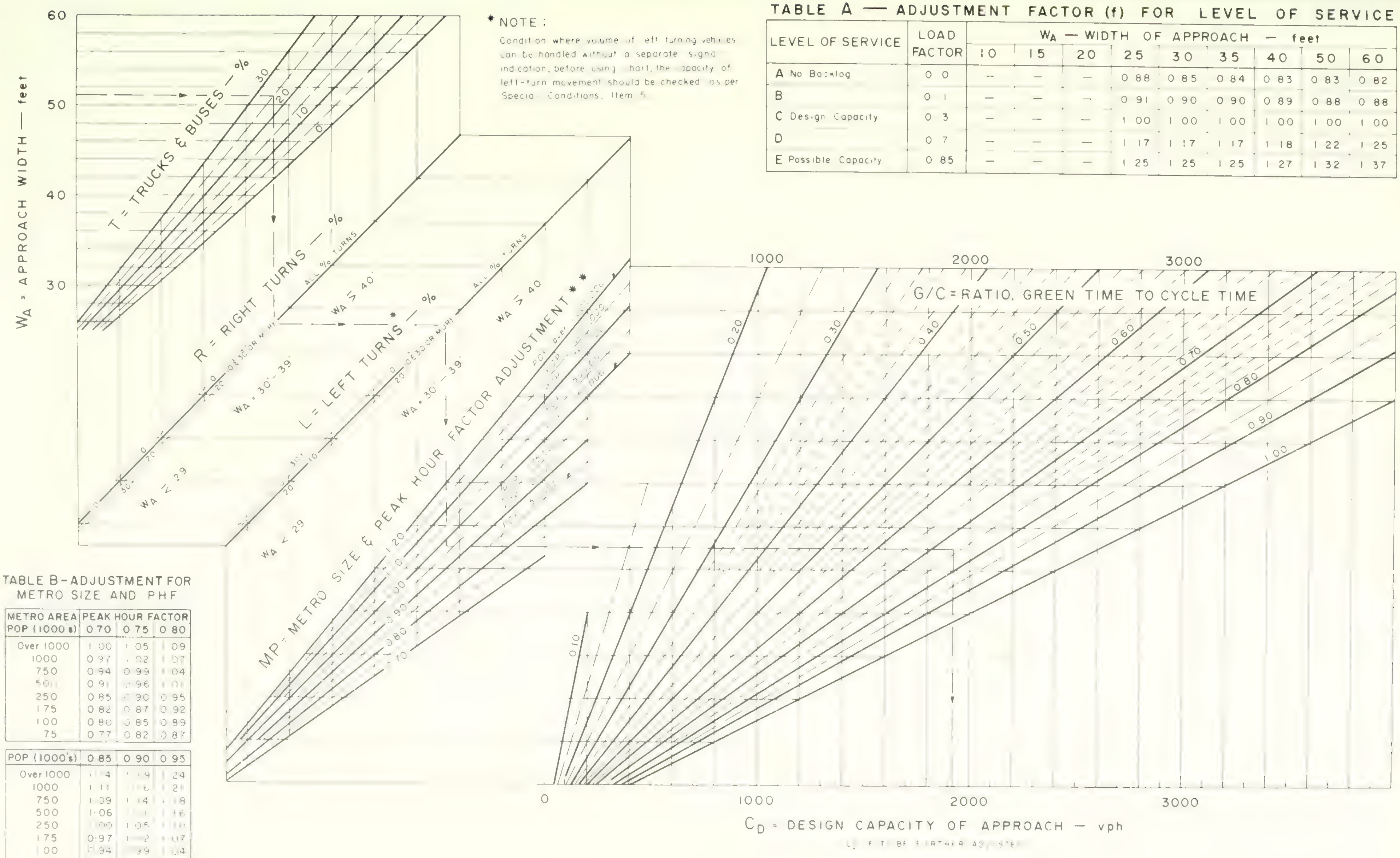

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	1.00	1.05	1.09
1000	0.97	1.02	1.07
750	0.94	0.99	1.04
500	0.91	0.96	1.01
250	0.85	0.90	0.95
175	0.82	0.87	0.92
100	0.80	0.85	0.89
75	0.77	0.82	0.87

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.19	1.24
1000	1.11	1.16	1.21
750	1.09	1.14	1.18
500	1.06	1.11	1.16
250	1.00	1.05	1.10
175	0.97	1.02	1.07
100	0.94	0.99	1.04
75	0.91	0.96	1.01

* * Use Table B if PHF is known [illegible]

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET - PARKING BOTH SIDES - O.B.D.

CHART 13

considered a part of the through movement, in which case $R=0\%$, would be used in the chart solution.

On the intersection approach of a 1-way facility where the turning conditions are as described above, either the right- or left-turning movement, or both, may be considered a part of the through movement. Such conditions are likely to occur at high-type, channelized intersections where turning movements can be accommodated as efficiently as through movements.

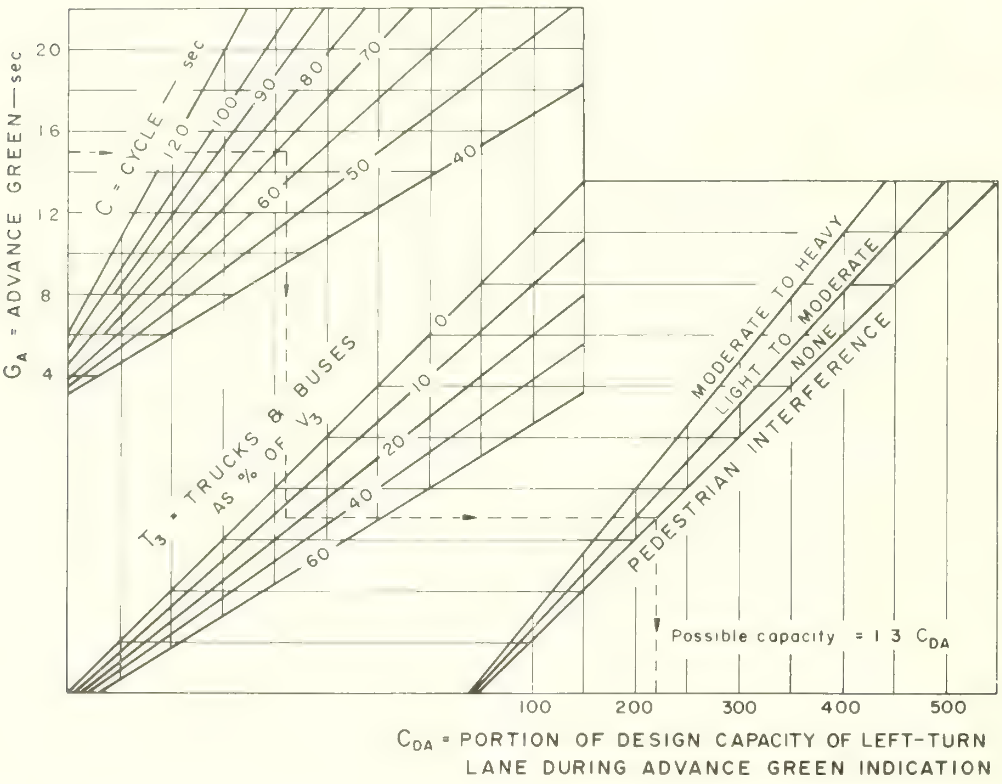

Figure 10.—Left-turn lane capacity on advance green indication.

Left-Turn Lane on Advance Green Indication

Left-turning movements on 2-way facilities that cannot be fully accommodated in the face of opposing traffic under normal 2-phase control require a separate signal indication. The control may be in the form of a third phase devoted to the left-turning movement. This type of signal arrangement is particularly advantageous when the opposing left-turning movement is also relatively high in volume so that both turning movements can be accommodated simultaneously on the same third phase.

When a left turning movement requires a separate signal indication, but the opposing left turn is sufficiently light so that it does not require a separate indication, an advance green period may be employed for the first left-turning movement to meet the capacity requirements. The advance green may be in the form of a separate green arrow introduced at the beginning of and simultaneously with the circular green indication for the through movement. The opposing movement is stopped by a red indication while the green arrow is in force. When the green arrow is turned off without using an amber period, the opposing movement is allowed to move and the regular circular green designation on the approach continues unabated. Thus, the left-turning movement is not only accommodated during the green arrow indication, but is allowed to move during the remaining regular green period on the approach as well. The additional left-turning vehicles that can be handled during this latter period depend on the volume of opposing traffic as for any left turn operated without a separate signal indication.

Another form of advance green indication is a flashing green, used effectively in parts of Canada. The initial flashing period on a regular green indication, during which both through and left turns move on the approach, is equivalent to the green arrow period previously described. When the flashing stops and the green becomes steady, the left turning traffic is under the influence of the opposing through traffic. This form of control, when a full third phase is not required simultaneously to accommodate the opposing left-turning movements, provides considerable efficiency because of the short advance green interval, usually 6 to 12 seconds long, the no amber period at termination of advance green, and the opportunity for left-turning movement to continue during the remaining green phase on the approach. The total number of left-turning vehicles that can be handled by a signal control incorporating an advance green indication may be determined as follows (The *Manual* does not include data or analyses of intersections using advance green indication for left turns; the following material was developed to fill this void in design and operation of arterial streets.)

(1) *Advance Green Period*—Use the nomograph in figure 10 to determine the number of vehicles accommodated per hour at design capacity during the advance green indication.[3]

(2) *Remaining Regular Green Indication*—Use charts 17-A and 17-B in which G/C is exclusive of the advance green to determine the number of vehicles accommodated per hour at design capacity.

(3) *Total Design Capacity of Left-turn Lane*—Sum of design capacities in (1) and (2).

(4) *Total Possible Capacity of Left-turn Lane*—Total design capacity in (3) multiplied by 1.3.

Problem 32

If in problem 20 an advance green indication was used for the left-turning movement, determine the total length of green interval required during the a.m. peak to accommodate the traffic demand at design capacity on the east approach (see figure 5).

Solution: It has already been determined in problem 20 that the number of left-turning vehicles that can be handled at design capacity during the full green interval (2-phase control) on the east approach is 80 v.p.h. with a cycle of 72 seconds (chart 17-B). If an advance green is introduced, the number of vehicles turning left that would have to be accommodated by the advance green is (190—80) = 110 v.p.h. Enter at bottom of nomo-

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[3] The nomograph is based on available but limited research. Vehicles accommodated on left-turn lanes under completely loaded conditions (1.00 load factor, representative of possible capacity), as reported from operational studies by D. G. Capelle and C. Pinnell (*5*), can be expressed as: $N=0.477\ (G-1.6)$, where N=number of passenger cars per cycle, and G=green interval in seconds. These findings are similar to those of previous research by R. M. Bartle, V. Skoro and D. L. Gerlough (*6*). Dividing the above expression by 1.3, ratio of possible capacity to design capacity previously established and substituting G_A as advance green for G, the formula becomes $N=0.38\ (G_A-1.6)$. This relation was tested against results of operational studies, carried out by F. J. Sanson (1962) and by V. Mitranic (1967) in Toronto, of left-turning movements on advance green indication. The studies by Capelle and Pinnell, when reduced to design capacity, compared favorably with the latter studies which were indicative of normal or average peak-hour operations, particularly for G_A values of 6 to 12 seconds. The nomograph in figure 10 is based on $N=0.38\ (G_A-1.6)$, with adjustments for trucks and pedestrian interference as previously established.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	—	—	—	0.88	0.85	0.84	0.83	0.83	0.82
B	0.1	—	—	—	0.91	0.90	0.90	0.89	0.88	0.88
C Design Capacity	0.3	—	—	—	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	—	—	—	1.17	1.17	1.17	1.18	1.22	1.25
E Possible Capacity	0.85	—	—	—	1.25	1.25	1.25	1.27	1.32	1.37

* NOTE

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5

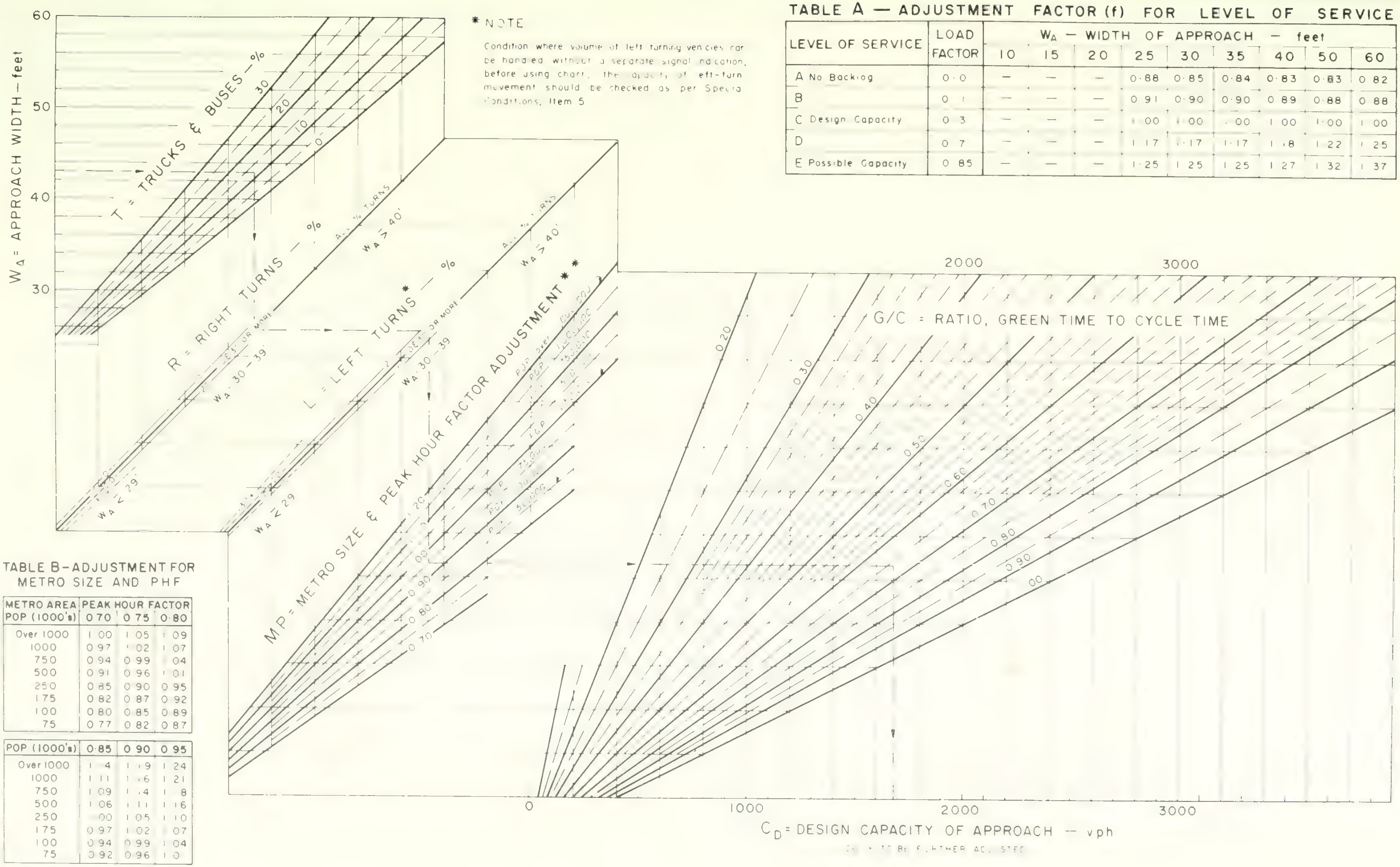


TABLE B-ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP (1000's)	PEAK HOUR FACTOR 0.70	0.75	0.80
Over 1000	1.00	1.05	1.09
1000	0.97	1.02	1.07
750	0.94	0.99	1.04
500	0.91	0.96	1.01
250	0.85	0.90	0.95
175	0.82	0.87	0.92
100	0.80	0.85	0.89
75	0.77	0.82	0.87

POP (1000's)	0.85	0.90	0.95
Over 1000	1.14	1.19	1.24
1000	1.11	1.16	1.21
750	1.09	1.14	1.18
500	1.06	1.11	1.16
250	1.00	1.05	1.10
175	0.97	1.02	1.07
100	0.94	0.99	1.04
75	0.92	0.96	1.0

* * Use Table B if PHF is known to find adjust. factor, otherwise [illegible]

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET—PARKING BOTH SIDES—RESIDENTIAL AREA
CHART 14

14

RES.

graph in figure 10 with a volume of 110 v.p.h. and proceed up and to the left using the conditions: no pedestrians, $T_3=11\%$ and $C=72$; read $G_A=9$ seconds, or $G_A/C=9/72=0.13$.

The G/C required for the through-plus-right movement on the east approach is 0.43, given in chart 1 and shown in problem 20. If $G/C=0.43$ is also required on the west approach, the total green time requirements on the east approach would be $G/C=0.43+0.13=0.56$ in this solution, compared with 0.70 in problem 20. On the other hand, if G/C of 0.30 or less was required on the west approach, the comparison would be 0.43 with 0.70; or the available time for amber and for green on the north-south street would be G/C of 0.57 in this solution and G/C of 0.30 in the solution of problem 20.

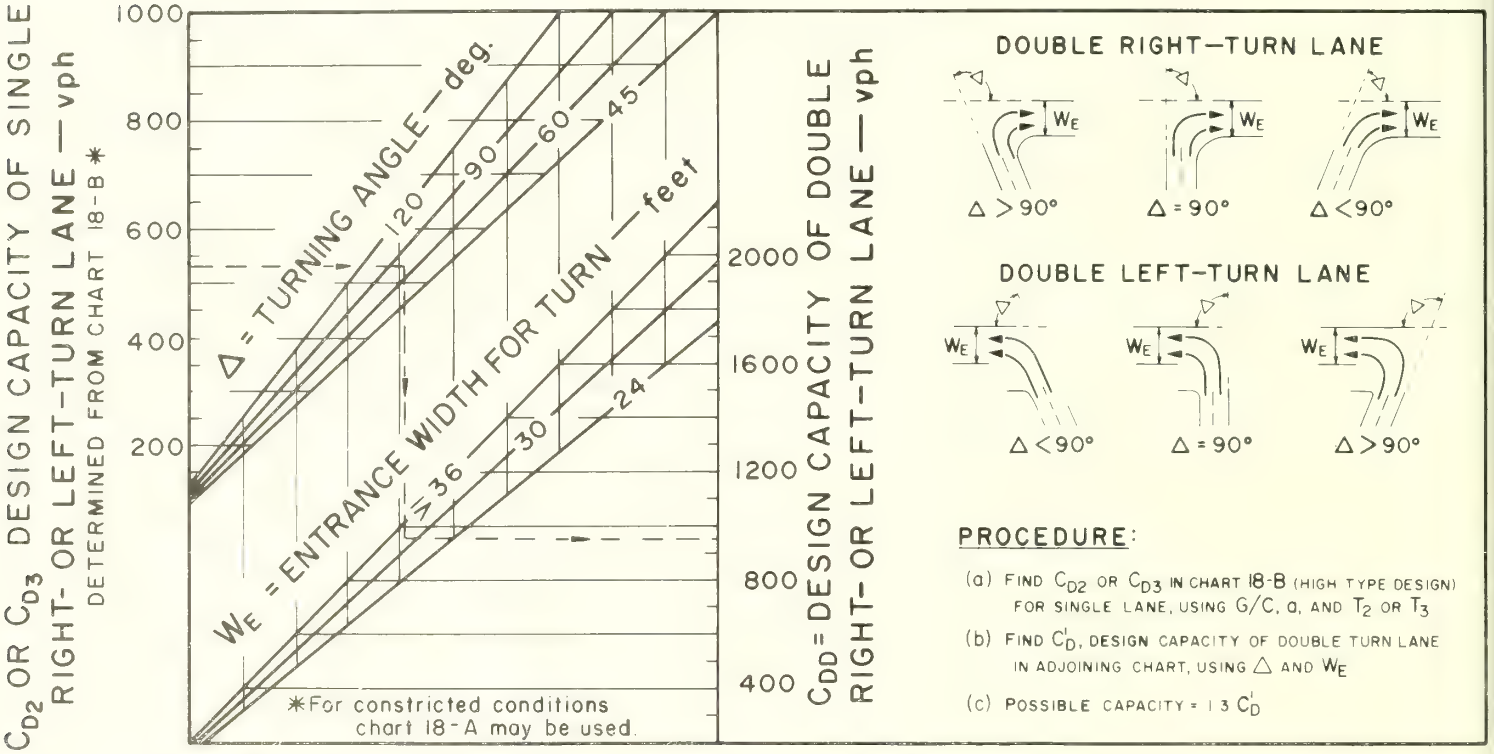


Figure 11.—Capacity of double turning lanes.

Right-Turning Movement—Continuous, Controlled by Yield Sign, or Permitted on Red After Stop

Special right-turning controls for continuous right-turning movement or for right-turning movement either controlled by yield signs or permitted on red after stop, are not covered in the *Manual*. The following suggested procedures are approximations developed for design purposes only. Right turns controlled by yield sign, and those permitted on red after stop, assume continuously high loading on the street being entered by right turns. The values given, therefore, are considered to be safe for design.

(1) *Continuous right-turning movement on an exclusive lane with an auxiliary lane on the crossroad and turning speeds upwards of 15 m.p.h.* Design capacity is estimated to be 1,200 passenger cars per hour (p.c.p.h.), or $1,200\div(1+t)$, where t is the proportion of trucks in the turning movement expressed decimally, that is, percentage of trucks divided by 100. Possible capacity is approximately 1.3 times design capacity.

(2) *Right-turning movement controlled by yield sign at the point of crossroad entry, and not by the intersection signal, on an exclusively right-turn lane with channelization*—Capacity depends on the availability of gaps on the crossroad. Because gap availability is difficult to predict, it is assumed that during peak periods there will be little, if any, opportunity for the right-turning movement to merge when the crossroad has a green indication. For design purposes it is assumed that an average of 3 vehicles can merge per cycle in addition to those handled during the green indication on the approach. These merges are considered to take place as a spill-over from the green indication on the approach, plus an occasional entry during the green indication on the crossroad; the several vehicles per cycle are regarded as including an average proportion of trucks. The design capacity, therefore, is that found in chart 17-D for the G/C ratio on the approach as a whole, plus $10,800/C$ v.p.h. Possible capacity is 1.3 times design capacity.

(3) *Right-turning movement on exclusive lane controlled by signal with turn permitted on red*—The condition is similar to (2), but less efficient. To find design capacity, use chart 17-C or 17-D, plus 2 vehicles per cycle or $7,200/C$ v.p.h. Possible capacity is 1.3 times design capacity.

Capacity Controlled by Intersection Exit

Generally the capacity of the approaches controls the capacity of the intersection. At some locations, however, the width of traveled way beyond the intersection may be restricted or traffic may back up onto the intersection exit from the intersection ahead. The latter situation may be corrected by coordination of signals and by the use of lagging or leading green indications. Where the traveled way

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0.0	0.92	0.95	0.96	0.96	0.97	0.97	0.97	0.97	0.96
B Design Capacity	0.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
C	0.3	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
D	0.7	1.21	1.21	1.22	1.23	1.25	1.27	1.29	1.31	1.33
E Possible Capacity	0.85	1.28	1.28	1.28	1.30	1.32	1.35	1.38	1.41	1.44

EXAMPLE

Given

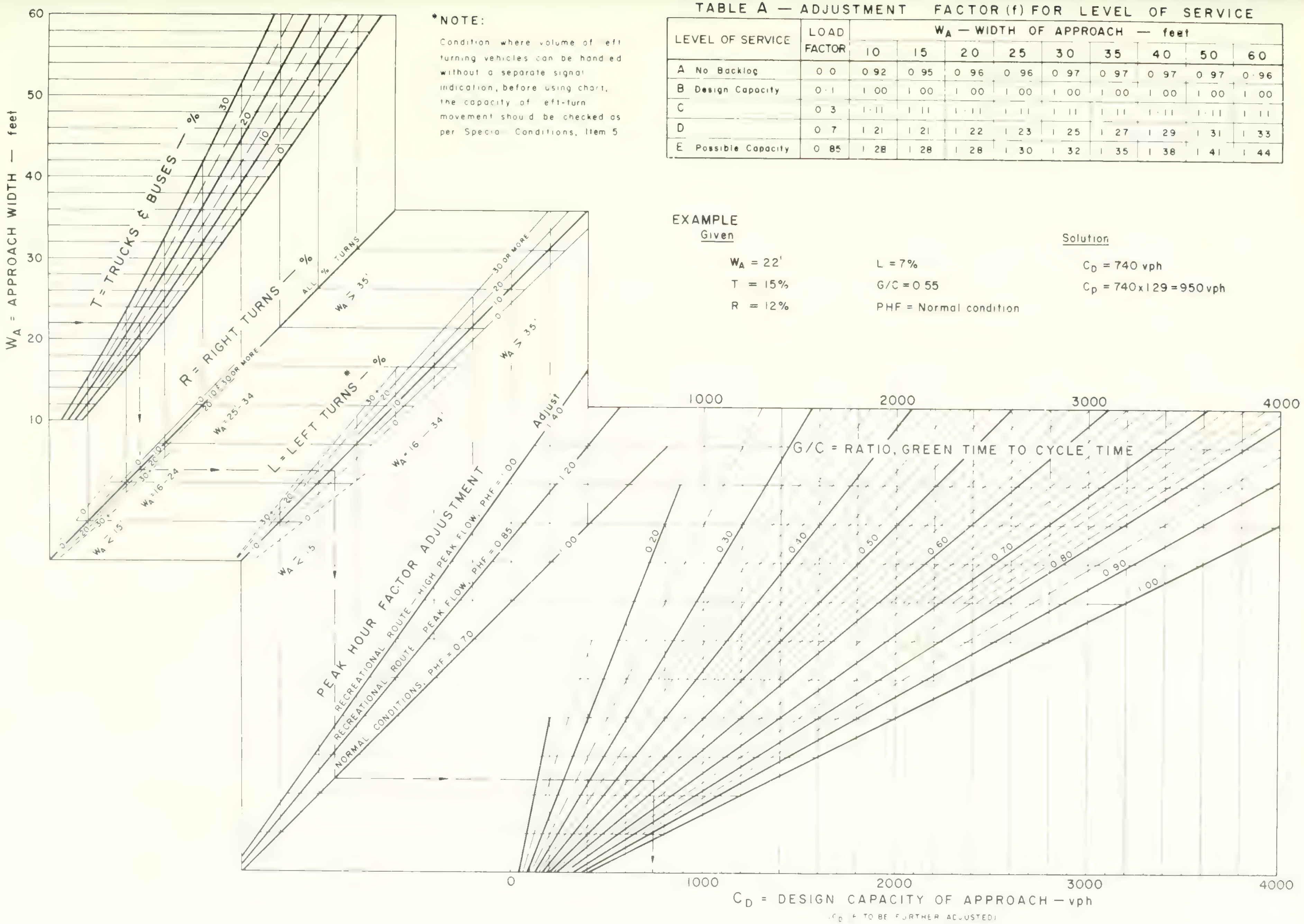
$W_A = 22'$
$T = 15\%$
$R = 12\%$
$L = 7\%$
$G/C = 0.55$
PHF = Normal condition

Solution

$C_D = 740$ vph
$C_P = 740 \times 1.29 = 950$ vph

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication, before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
RURAL TWO-WAY HIGHWAY — NO PARKING
CHART 15

available for moving vehicles is narrower on the intersection exit than on the intersection approach, capacities may be estimated as follows: Enter charts 3–14 with W_A equal to the width of the exit. Exit width is measured from division line to curb on 2-way streets, and curb to curb on 1-way streets. Proceed through chart in usual manner, but use T percentage of trucks and buses in through movement only, $R=0\%$, $L=0\%$, MP=metro population, and G/C equivalent to that used on the approach; adjust for bus stop only if it is on the far side, using $L+R=0\%$.

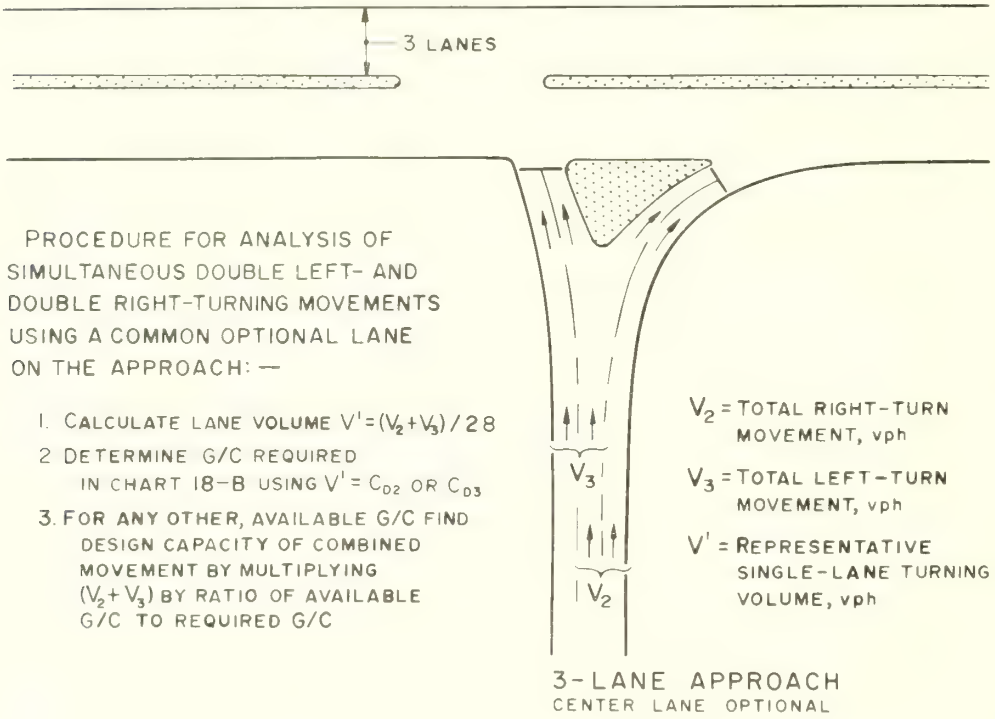

3 LANES
PROCEDURE FOR ANALYSIS OF SIMULTANEOUS DOUBLE LEFT- AND DOUBLE RIGHT-TURNING MOVEMENTS USING A COMMON OPTIONAL LANE ON THE APPROACH:—
1. CALCULATE LANE VOLUME V'=(V2+V3)/2.8
2 DETERMINE G/C REQUIRED IN CHART 18-B USING V' = CD2 OR CD3
3. FOR ANY OTHER, AVAILABLE G/C FIND DESIGN CAPACITY OF COMBINED MOVEMENT BY MULTIPLYING (V2+V3) BY RATIO OF AVAILABLE G/C TO REQUIRED G/C
V3
V2
V2 = TOTAL RIGHT-TURN MOVEMENT, vph
V3 = TOTAL LEFT-TURN MOVEMENT, vph
V' = REPRESENTATIVE SINGLE-LANE TURNING VOLUME, vph
3-LANE APPROACH
CENTER LANE OPTIONAL

Figure 12.—Combined double left-turning and double right-turning movements with optional lane.

2-Lane Turning Movements

Where the capacity of a right- or left-turn lane is insufficient to handle the demand volume, the use of a double turning lane should be considered. This may be on two exclusive lanes, or on one exclusive lane and a second optional lane; the optional lane simultaneously would be accommodating a through or a right-turning movement. *Only the general case of the double turning lane is covered in the Manual. A rational method for evaluation of capacities of 2-lane turning movements with different angles of turn and entry widths on intersecting road is developed here for design purposes.*

Double right- or left-turn lane on 2 exclusive lanes

The capacity of a double turning lane may be determined in accordance with the procedure outlined in figure 11 using chart 18-B in combination with the nomograph in figure 11. The procedure is based on the following: According to the *Manual*, at a usual intersection the second or outer lane of a 2-abreast turning movement is capable of handling a service volume or capacity of 0.8 times the value for a turning lane indicated in charts 17 and 18, or an average per lane of 0.9 times the single-lane capacity. This pertains to a turn at or near 90 degrees and with sufficient width available for each line of vehicles throughout the turn. The capacity of the double turning movement is sensitive to the angle of turn, so that for acute-angle turns of more than 90 degrees, the capacity is reduced, and for obtuse-angle turns of less than 90 degrees, the capacity is increased. This sensitivity to angle of turn may be expressed by a factor of less than or more than 0.90.

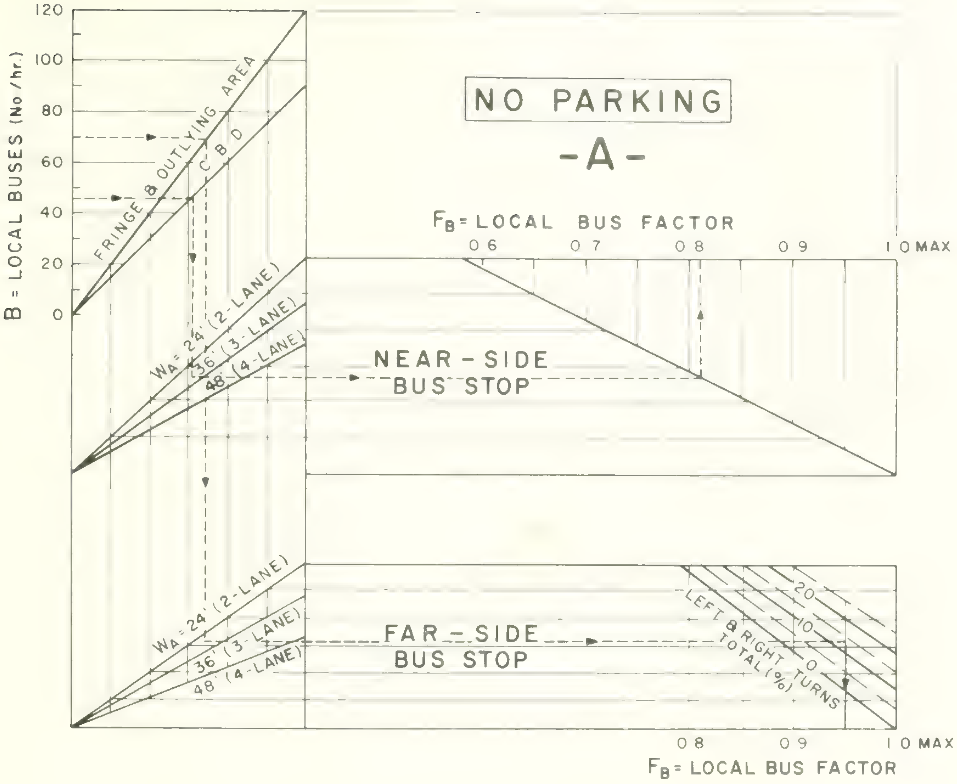
At some angle of turn flatter than 90 degrees, the full efficiency of the second lane will be achieved so that the 0.90 factor will become 1.00. Moreover, at an extremely flat angle of turn with channelization, the capacity of the movement will approximate the capacity of a usual 2-lane intersection approach as represented in charts 3 and 4 and 7–9. To achieve some correlation between turning lane capacities and regular intersection approach capacities, an adjustment factor of 1.10 has been selected for angles of turn of 45 degrees or less. For an intermediate angle of 60 degrees, a factor of 1.00 is designated. On the other hand, for an acute angle of 120 degrees the indicated adjustment factor is 0.80. Because a double-turn movement is usually designed to handle relatively large turning volumes free of pedestrian interference, the capacity for the high-type design, chart 18-B, is generally applicable. The value for a single lane from chart 18-B, therefore, is first determined and then multiplied by a factor of 0.80 to 1.10, depending on the turning angle, to produce the design capacity of a double right- or left-turn lane.

Another characteristic of a 2-abreast turning movement that affects its traffic carrying capabilities is the lateral space available upon turning, which determines whether the driver feels a sense of restriction or freedom. At the beginning of the turn the capacity is controlled by the width of approach lanes, 10, 11, or 12 feet. Turning lanes wider than 12 feet on the approach are considered to have the same capacity as 12-foot lanes. Upon completing the turn, the capacity is controlled by the width available for the 2 lanes at the far end of the turn or by the entrance width for the turn on the street receiving the turn, as represented by W_E on the right-hand side of figure 11. Experience shows that approximately a 3-lane width is required at this point to assure the freedom of movement necessary to realize the full capacity of the double turn. An entrance width of 36 feet or more is considered ideal; the adjustment factor therefore is 1.00. Values for W_E of 30 and 24 feet are assumed to yield smaller capacities, expressed by adjustment factors of 0.90 and 0.80 respectively. These factors combine as a product with those given previously for the angle of turn, so that for $\Delta=45°$ and $W_E=36$, the composite factor is $1.10\times1.00=1.10$, and for $\Delta=120°$ and $W_E=24$ the composite factor is $0.80\times0.80=0.64$. These factors are applied to twice the design capacity of a single lane and are indicative of the range of values applicable to design of intersections.

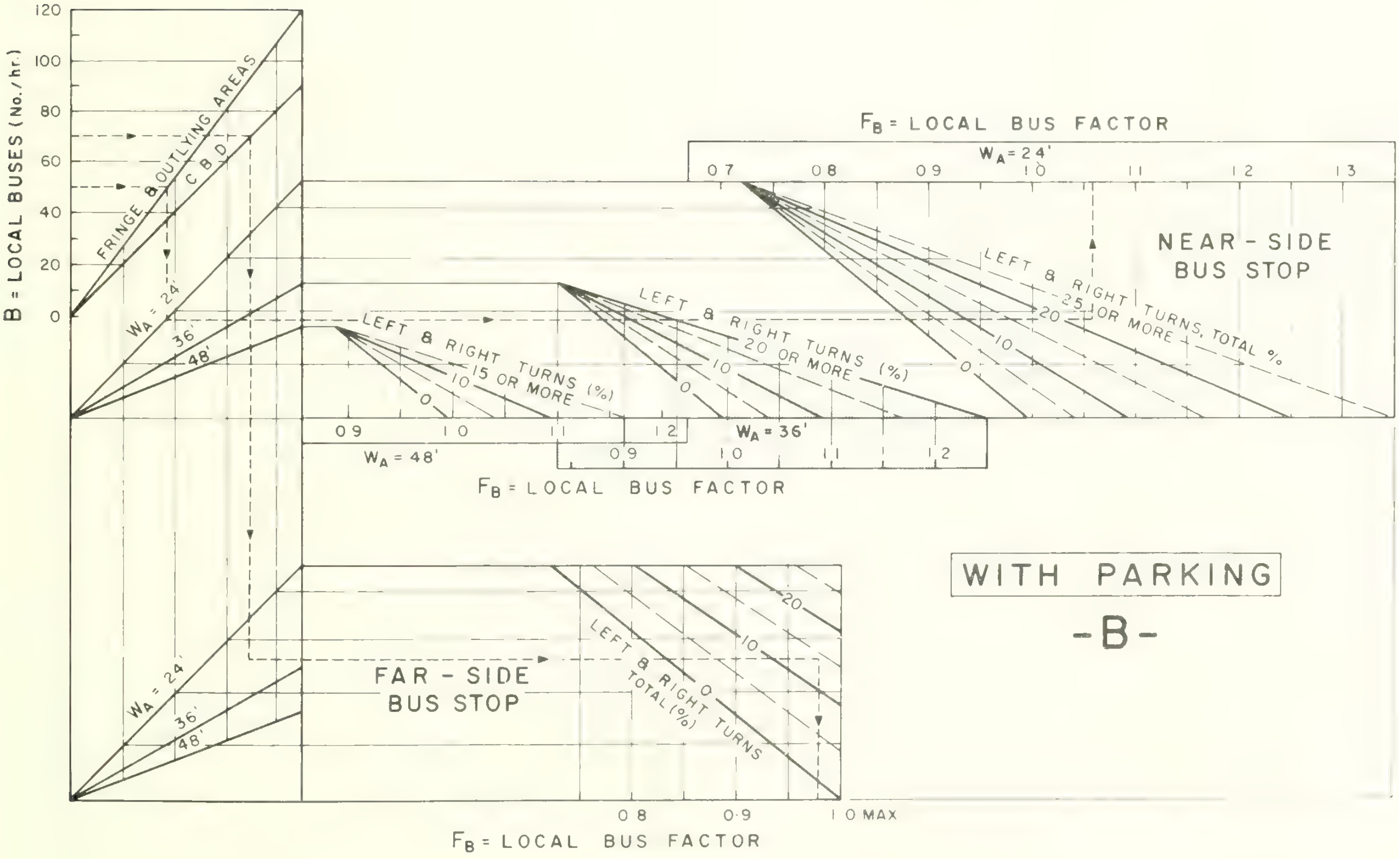
Capacity relations as affected by the turning angle, Δ, and the cross-street entrance width, W_E, are shown in the nomograph. Thus, by using the design capacity for a single lane in chart 18-B, the design capacity (C_{DD}) of the double right- or left-turn lane is found directly in the chart of figure 11.

Problem 33

What is the design capacity and possible capacity of a double left-turning lane 22 feet wide on the approach, two 11-foot lanes, where the angle of turn is 75 degrees and the traveled way, W_E, on the cross street receiving the turn is 33 feet wide? Other conditions are: trucks comprise 17% of the turning movement, and the G/C available for the movement is 0.60.



NOTE:
Local buses are to be included in determining overall percentage of trucks and buses (T) on Charts 3-14

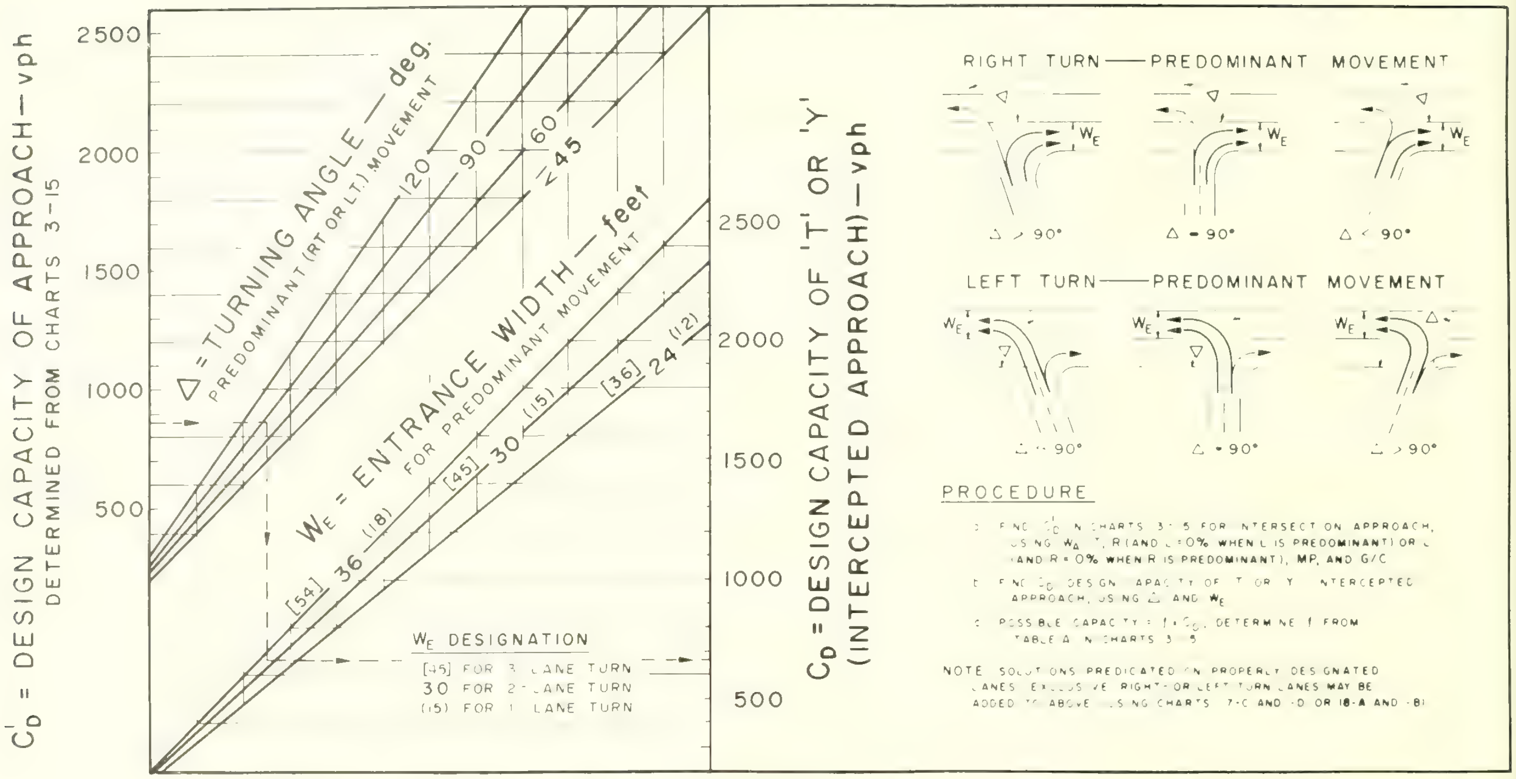


DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

LOCAL BUS FACTOR

FOR USE IN CONJUNCTION WITH CHARTS 3 – 14

CHART 16



PROCEDURE

(a) FIND C'_D IN CHARTS 3–15 FOR INTERSECTION APPROACH, USING W_A, T, R (AND L = 0% WHEN L IS PREDOMINANT) OR L (AND R = 0% WHEN R IS PREDOMINANT), MP, AND G/C

(b) FIND C_D, DESIGN CAPACITY OF 'T' OR 'Y' INTERCEPTED APPROACH, USING Δ AND W_E

(c) POSSIBLE CAPACITY = $f \times C_D$. DETERMINE f FROM TABLE A IN CHARTS 3–15

NOTE: SOLUTIONS PREDICATED ON PROPERLY DESIGNATED LANES. EXCLUSIVE RIGHT- OR LEFT-TURN LANES MAY BE ADDED TO ABOVE USING CHARTS 17-C AND -D OR 18-A AND -B.

Figure 13.—Capacity of T or Y intersections.

Solution: The design capacity of single left-turn lane in Chart 18-B, using $G/C = 0.60$, $a = 11$ and $T_3 = 17\%$, is $C_{D3} = 530$ v.p.h. Entering with this value in the chart of figure 11, and applying $\Delta = 75$ and $W_E = 33$, the design capacity of the double left-turn lane is found to be $C_{DD} = 950$ v.p.h. Possible capacity $C_{PD} = 950 \times 1.30 = 1{,}240$ v.p.h.

2-abreast turning movement on 1 exclusive lane and on second optional lane

To achieve this, the intended operation should be designated by appropriate pavement marking and signing. Where the optional lane is shared by turning and through traffic, the following procedure should be followed:

(1) Determine the design capacity of the exclusive turning lane (C_{D2} or C_{D3}) in chart 18.

(2) Deduct the design capacity of the turning lane from the total turning volume $(V_2 - C_{D2})$ or $(V_3 - C_{D3})$.

(3) Determine the design capacity of the remainder of the approach roadway (W'_A) of which the optional lane is a part.

(4) The through volume plus $(V_3 - C_{D3})$ or $(V_2 - C_{D2})$ should not exceed the design capacity for approach width W'_A.

Where the optional lane is shared by a left- and a right-turning movement at a channelized intersection, as is often the case on ramp terminals of diamond and parclo interchanges, the procedure for analysis is outlined in figure 12. The formula shown in the figure allows for determination of the required G/C to accommodate the combined V_2 and V_3 volumes at design capacity by the use of chart 18-B. The formula is predicated on the condition where each of the 2 outer lanes discharges the usual volume of a turning lane at design capacity, while the middle lane discharges its share of the combined remaining left- and right-turning traffic at 0.8 design capacity. This assumes adequate radius and ample width for the double right-turning movement and a turning angle of 90 degrees or less with fully adequate exit width for the double left-turning movement. The design capacity of the combined movement for any other available G/C may be found by multiplying $V_2 + V_3$ by the ratio of available G/C to required G/C.

T and Y Intersections

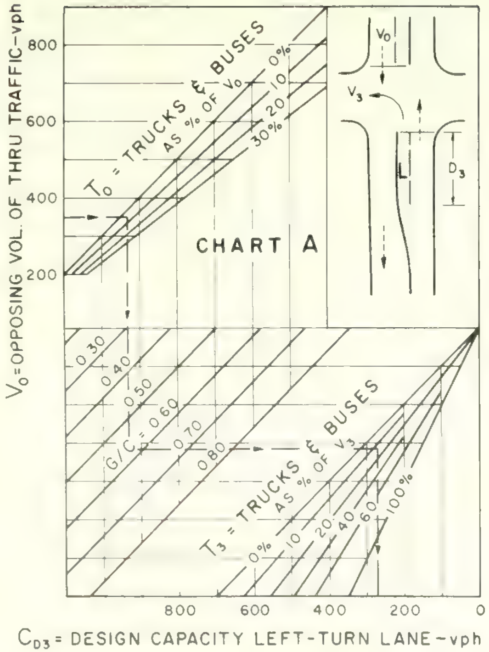
The capacity of the approach of an intercepted street at T and Y intersections, as the east approach in figure 14, may be obtained in accordance with the procedure outlined in figure 13, using charts 3–15 in combination with the nomograph in figure 13. Because the capacity studies reported in the *Manual* do not include direct measurements at T and Y intersections, a rational method was developed using the results from intersection approaches for the usual 4-leg intersections. As for the double turning lanes, the capacity of the intercepted approach at T and Y intersections is affected by the angle of turn and the entrance width for the turn on the street joined. Also, whether the right-turn volume or the left-turn volume is the predominant movement should be considered.

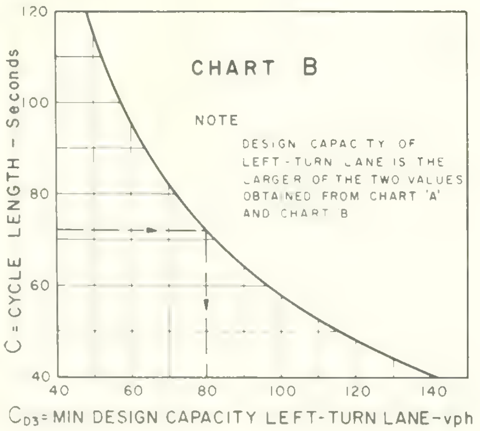
Because all traffic on the intercepted approach is turning, the total discharge from the approach—unless the angle of turn for the predominant movement is very favorable—would be less than for the approaches on regular intersections indicated in charts 3–15. Obviously, a double left-turn lane at a regular intersection and a 2-lane predominant left-turn movement on a T intersection, for the same width and angle controls, would have similar capacities. Results for the two conditions should be correlated, which is accomplished by equating, approximately, the capacities of a double left-turn lane with the capacities of a 2-lane intercepted approach on T intersections with 100% of the traffic turning left.

Averages of design and possible capacities of double turning lanes for angles of 120, 90, 60, and 45 degrees were compared, as a base, with averages of design and possible capacities of 2-lane approaches from charts 4 and 9 for an intermediate range of metro area sizes represented by 500,000 population. Ratios of the two sets of values for the different angles of turn, serving as adjustment factors, were found to be as follows:

17

TURN'G. LANES





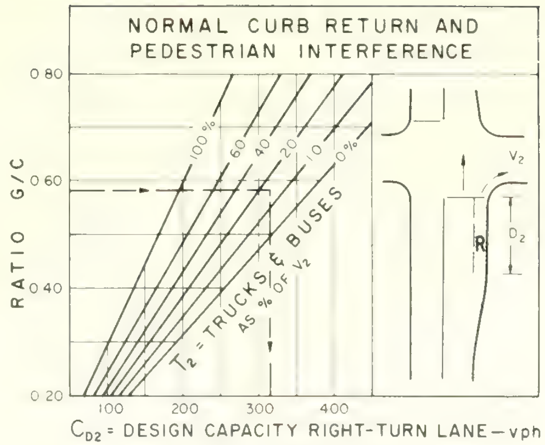


CHART C

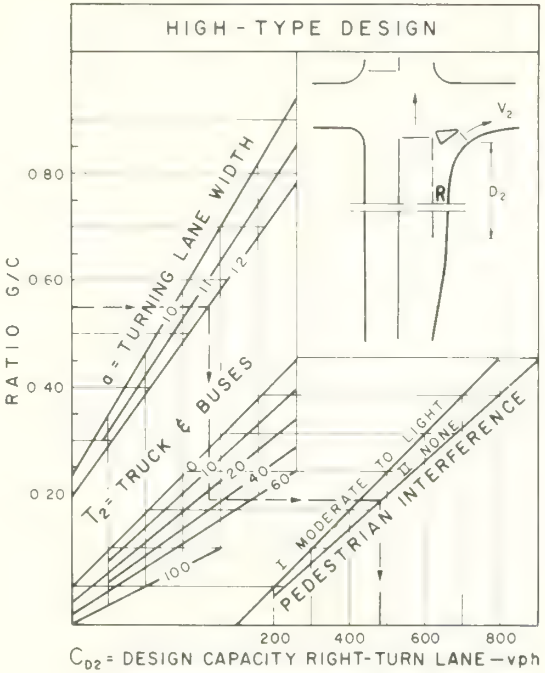
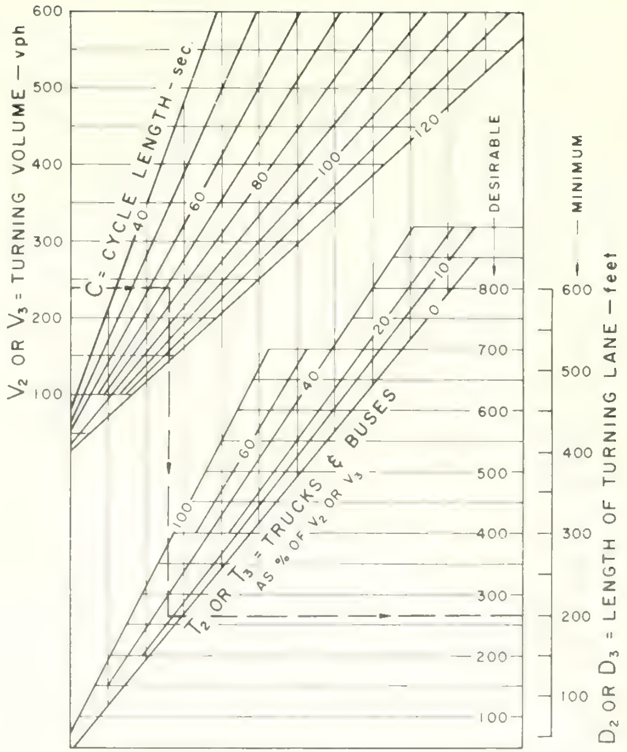


CHART D



LENGTH OF RIGHT- OR LEFT-TURN LANE

CHART E

NOTE

- CHARTS A AND B APPLY TO TWO-WAY FACILITIES ONLY, IN URBAN AS WELL AS IN RURAL AREAS
- CHARTS C AND D APPLY TO RIGHT-TURN LANES ON TWO-WAY STREETS AND TO BOTH RIGHT- AND LEFT-TURN LANES ON ONE-WAY STREETS
- CURVE II OF CHART D GENERALLY IS NOT APPLICABLE TO CBD
- IN RURAL AREAS (FOR SERVICE LEVEL B) MULTIPLY VALUES IN CHART D BY 0.90
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT TURN LANE MULTIPLY CHART DESIGN CAPACITY BY 1.30

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

SEPARATE RIGHT- AND LEFT-TURN LANES

NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT

CHART 17

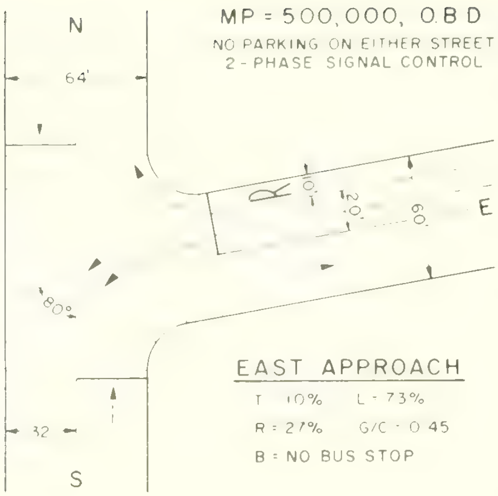


Figure 14.—Problem 34 illustrated.

Δ, degrees, for predominant movement	45	60	90	120
Adjustment factor, rounded	1.00	0.90	0.80	0.70

The adjustment factor of 1.00 indicates a capacity on the intercepted approach (T or Y intersection) equivalent to the full capacity of approach on a regular 4-leg intersection. Thus, for a turning angle of 45 degrees or less for the predominant movement, the directness of the turn is assumed to yield capacities of the order given in charts 3–15. The preceding apply when the intersection approach is not deterred by the intersection exit; that is, the width of traveled way, W_E, receiving a 2-lane predominant movement, is equal to 3 lanes. The same adjustment factors of 1.00, 0.90 and 0.80, used for double turning lanes, are applied here for W_E of 36, 30, and 24 feet, respectively. The factors developed for 2-lane approaches are considered to be applicable to other widths of approach and other conditions including parking. The relations for T and Y intercepted approaches are presented in the nomograph in figure 13.

Problem 34

What are the design and possible capacities of the intercepted street (east approach), from which traffic can turn only left and right into the north-south street, for the conditions indicated in figure 14?

Solution: In chart 4, using $W_A=20$, $T=10\%$, $L=0\%$, $R=0\%$, (because right turn is handled on separate right-turn lane), $MP=500{,}000$ population, and $G/C=0.45$, find $C_D=860$ v.p.h. Entering with this value in the chart of figure 13, and applying $\Delta=80°$ and $W_E=32$, the design capacity of the two lanes handling traffic to the left is $C_D=650$ v.p.h. The corresponding right-turn volume$=650\times 27/73=240$ v.p.h. The design capacity of the right-turn lane from chart 17-C is 260 v.p.h. The volume on the approach that can be accommodated at design capacity$=650+240=890$ v.p.h. The possible capacity of the left-turn movement is $650\times 1.20=780$ v.p.h. The corresponding right-turn volume$=780\times 27/73=290$ v.p.h. Possible capacity of right-turn movement is $260\times 1.30=340$ v.p.h. The volume on the approach that can be handled without exceeding possible capacity$=780+290=1{,}070$ v.p.h.

Multiple-Type Intersections

The capacity of any form of signalized intersection, regardless of the number of approach roads and extent of channelization, can be obtained from the charts by examining each approach road separately. The design of complex intersections, particularly those requiring multi-phase control, may necessitate some study and trial solutions before determining the final plan. Multiple intersections often present several possibilities in the pattern of operation and in the number and arrangement of signal phases. Such alternative arrangements may result in different geometric layouts affecting the shape and location of islands, widths of pavements, size of storage areas, and overall space requirements for the intersection. The geometric layout should be determined jointly with capacity analyses. Care should be taken to check the length and width of those traffic channels where vehicles will store during certain signal phases to preclude the condition of traffic backing up from one intersection point to another. Use of advance or lagging green indications and appropriate offsets to produce progressive movements may be necessary in this regard. Time-space diagrams often are valuable adjuncts to the design procedure.

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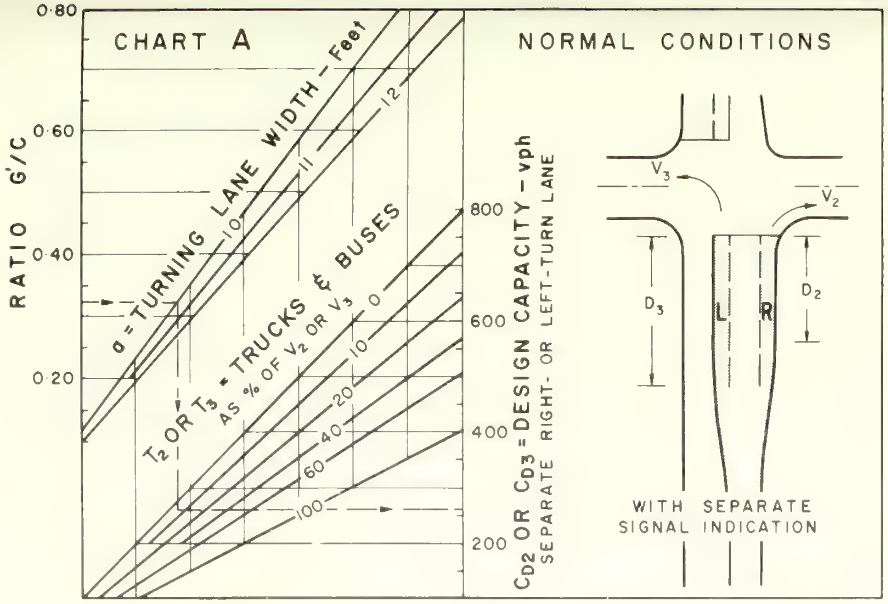
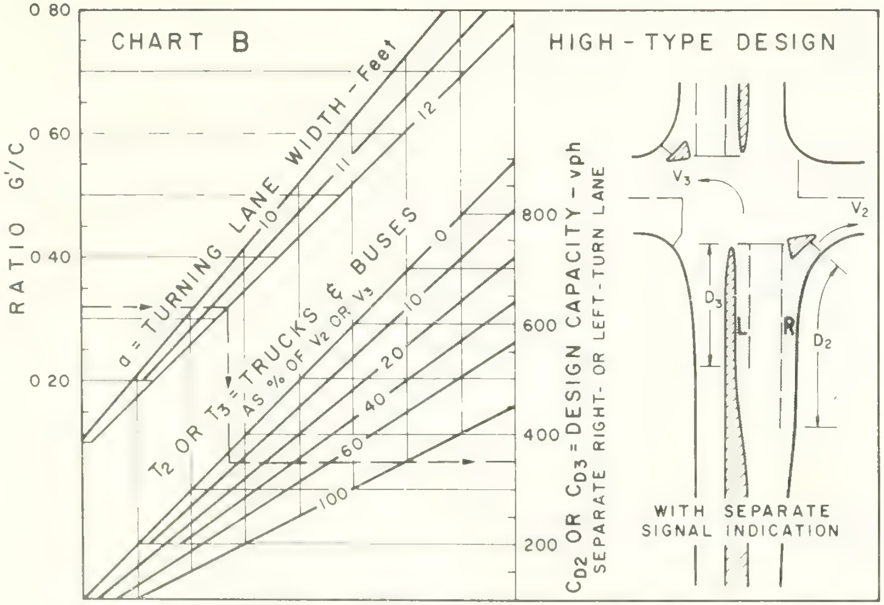
(3) *Design Capacity Charts for Signalized Street and Highway Intersections,* by Jack E. Leisch, *PUBLIC ROADS, A Journal of Highway Research,* vol. 26, No. 6, Feb. 1951, pp. 105–139.

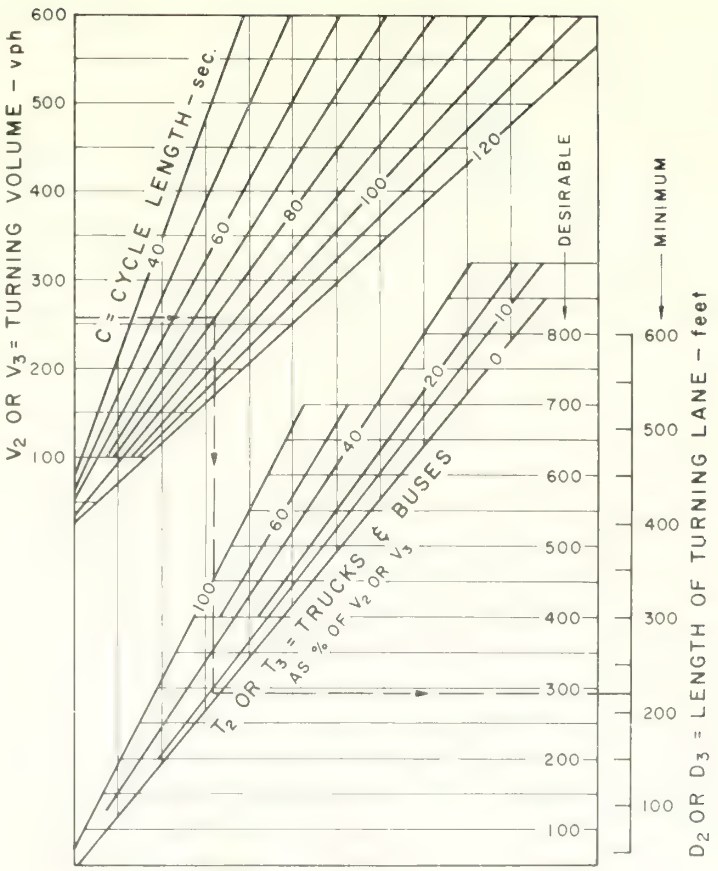
(4) *A Policy on Geometric Design of Rural Highways,* by American Association of State Highway Officials, 1965.

(5) *Capacity Study of Signalized Diamond Interchanges,* by Donald G. Capelle and Charles Pinnell, HRB Bulletin 291, Freeway Design and Operations, publication 866, 1961, pp. 1–25.

(6) *Starting Delay and Time Spacing of Vehicles Entering Signalized Intersection,* by Richard M. Barkle, Val Skoro, and D. L. Gerlaugh, HRB Bulletin 112, Effects of Traffic Control on Street Capacity, publication 365, 1956, pp. 33–41.







LENGTH OF RIGHT- OR LEFT-TURN LANE
CHART C

NOTE

- CHARTS A AND B APPLY TO BOTH TWO-WAY AND ONE-WAY STREETS.
- CHART B GENERALLY DOES NOT APPLY TO CBD
- IN RURAL AREAS (FOR SERVICE LEVEL B) MULTIPLY VALUES IN CHARTS A AND B BY 0.90
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE, MULTIPLY CHART DESIGN CAPACITY BY 1.3

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

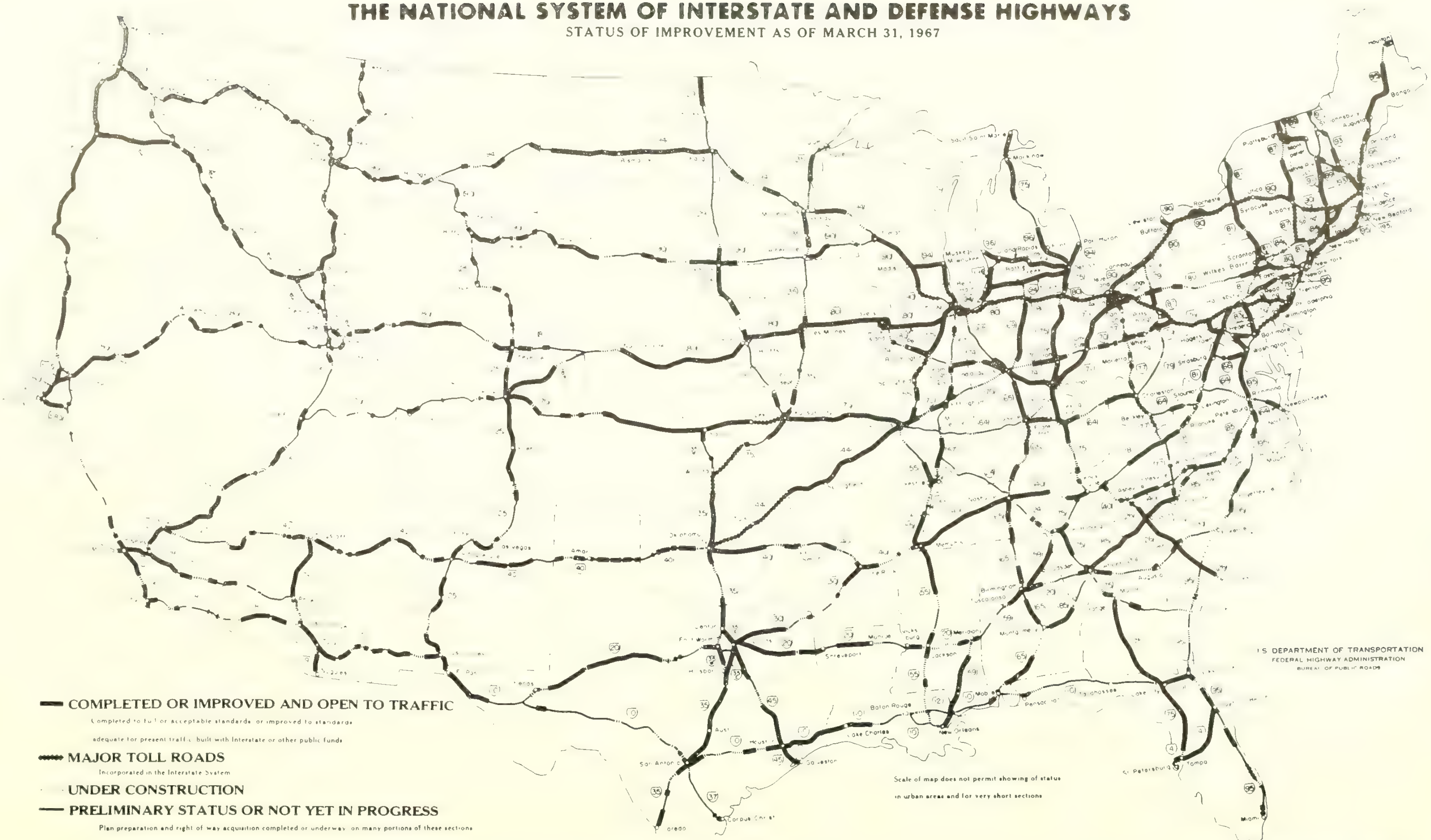
SEPARATE RIGHT- AND LEFT-TURN LANES

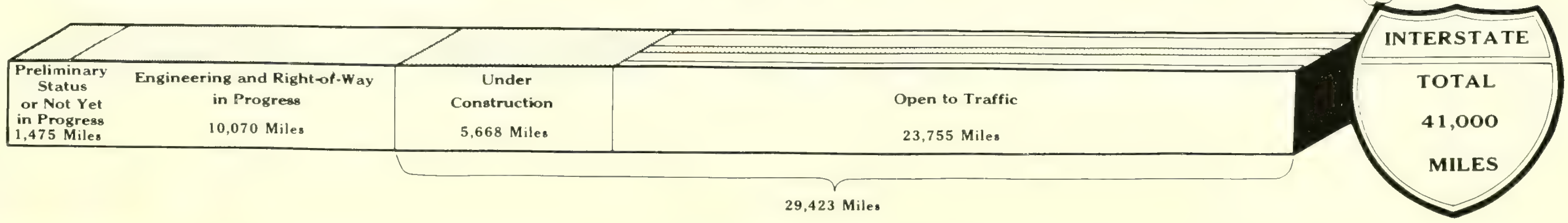
WITH SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENTS

CHART 18

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF MARCH 31, 1967


COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards or improved to standards adequate for present traffic built with Interstate or other public funds
MAJOR TOLL ROADS
Incorporated in the Interstate System
UNDER CONSTRUCTION
PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right of way acquisition completed or underway on many portions of these sections
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS
Scale of map does not permit showing of status in urban areas and for very short sections



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SPECIAL ISSUE

Design capacity charts for signalized intersections

Second installment—continued from vol. 34, No. 9

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Capacity Analysis Techniques for Design of Signalized Intersections

Installment No. 2

Sponsored by the
OFFICE OF ENGINEERING AND OPERATIONS
BUREAU OF PUBLIC ROADS

by [1] JACK E. LEISCH, Vice President and Chief Highway Engineer, DeLeuw, Cather & Co. of Canada Ltd.

PART 4—HIGH-TYPE FACILITIES AND INTERCHANGES

This is the second and final installment of an article in which procedures are presented for the graphic solution of capacity problems related to signalized intersections. The first installment was published in the August issue, vol. 34, No. 9, of PUBLIC ROADS.

The procedures are based on a set of charts consisting of 20 nomographs. Eighteen of the nomographs together with appropriate application procedures and sample problems were presented in the first installment. The other two nomographs and the remainder of the article are presented here.

The nomographic charts and procedures were devised by the author in 1950 to simplify the computational procedures of the 1950 Highway Capacity Manual. They were presented in PUBLIC ROADS in 1951 and were acclaimed by those concerned with intersection design. Since publication of the 1965 Highway Capacity Manual has provided a revised and comprehensive basis for capacity computations, the author in this article has again filled the need for a graphic procedure incorporating current knowledge. The original charts have been updated and new charts have been prepared to cover capacity procedures for which calculations previously required extensive application of judgment. The information presented provides a graphic procedure for the capacity analysis of most signalized street and highway intersections. Full discussion of the principles and procedures in the application of the charts in addition to sample problems have been included.

THE PROCEDURES AND CHARTS dealt with in previous parts of this article (PUBLIC ROADS vol. 34, No. 9) are also applicable to high-type facilities, including intersections designed to above-minimum standards that incorporate channelization. Sometimes such facilities accommodate relatively high-speed traffic characteristic of suburban and rural conditions. The at-grade ramp terminals of diamond and parclo interchanges generally are forms of high-type intersections. Whereas the problem solutions previously covered consider only one or two approaches to illustrate basic procedures and uses of charts, part 4 deals with the entire intersection—all approaches and a complete solution. Included is a standard computational form to facilitate analyses and a suggested format for a drawing or sketch showing the resulting geometric design, signal phasing, and a summary of volume-capacity relations.

Problem 35

The intersection indicated in figure 15 operates under congested conditions during peak periods, particularly between 5:15 and 6:15 p.m. It is to be reconstructed, not only to remove the present bottleneck but also to accommodate, at level of service C, the future traffic based on a 15-year projection. The north-south expressway at-grade, which has a design speed of 50 m.p.h., is to remain substantially the same. The east-west arterial, however, is to be improved basically to a 4-lane divided highway, using a design speed of 40 m.p.h., 12-foot lanes and a 16-foot median, with additional lanes, as required, at the major intersections. The percentages of trucks on the different approaches are N and S—6 percent, W—10 percent, and E—12 percent. A continuous right-turning movement is to be provided from W to S. Determine the geometrics for the improvement and the signal timing. Right-of-way is not a factor; moderate channelization is to be considered.

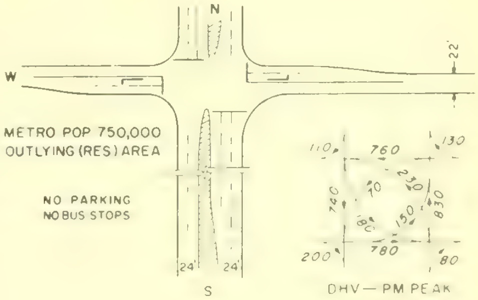

Figure 15.—Problem 35 illustrated.

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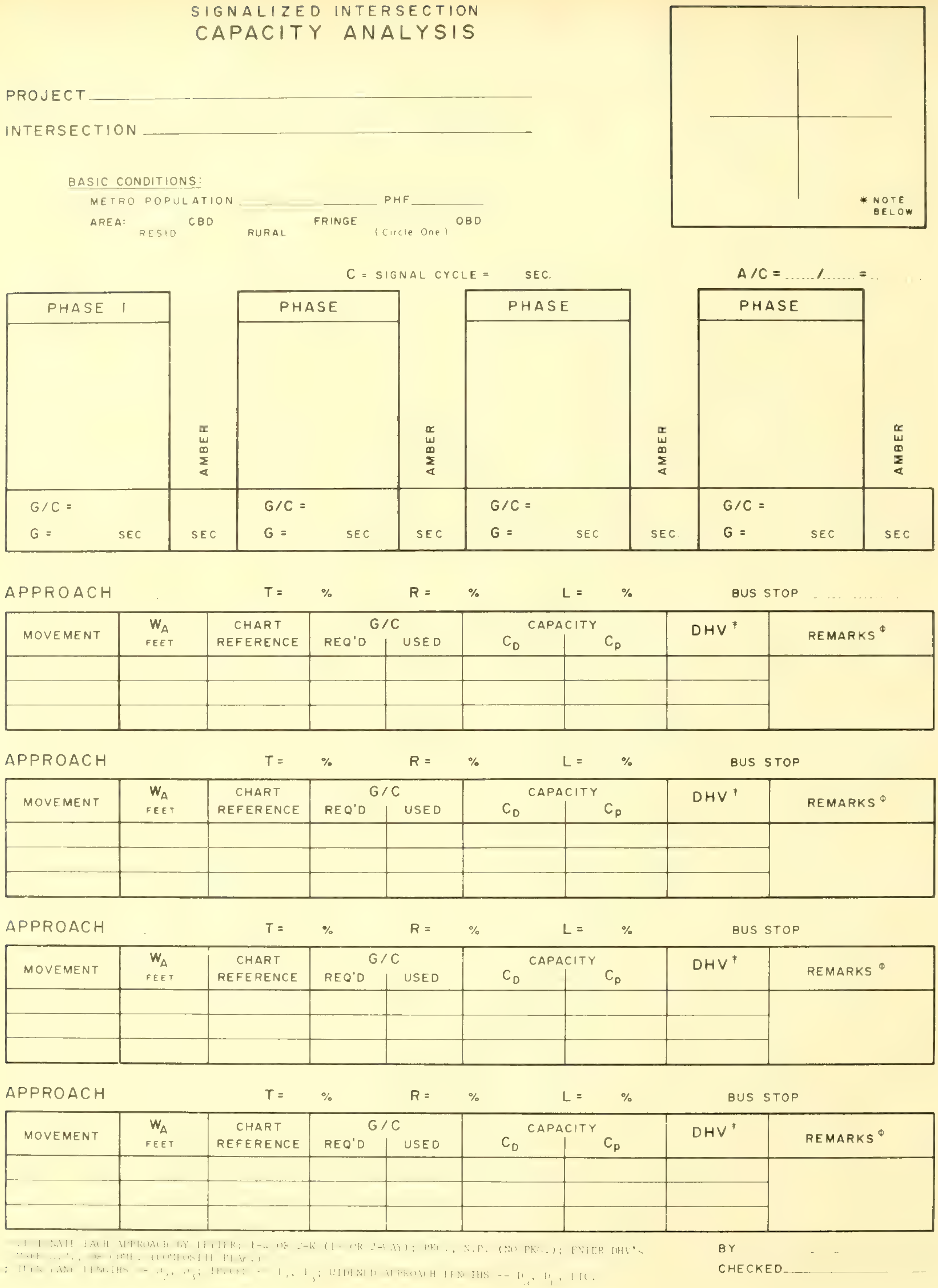
[1] Mr. Leisch was formerly Chief of Design Development Branch, Bureau of Public Roads. Mr Leisch acknowledges the assistance of DONALD W. LOUTZENHEISER, WILLIAM P. WALKER, and DONALD B. LEWIS of the Bureau of Public Roads who provided guidance during preparation of the material and reviewed the completed work. JOEL P. LEISCH and ARNE HAALAND of DeLeuw, Cather & Co. of Canada Limited also assisted in preparation of material and development of charts.



SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT ____________________

INTERSECTION ____________________

BASIC CONDITIONS:

METRO POPULATION ____________ PHF ________

AREA: CBD FRINGE OBD RESID RURAL (Circle One)

C = SIGNAL CYCLE = SEC.

A/C = ____/____ = __

PHASE 1	AMBER	PHASE	AMBER	PHASE	AMBER	PHASE	AMBER
G/C = G = SEC	SEC	G/C = G = SEC	SEC	G/C = G = SEC	SEC.	G/C = G = SEC	SEC

APPROACH T = % R = % L = % BUS STOP

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV †	REMARKS Φ

APPROACH T = % R = % L = % BUS STOP

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV †	REMARKS Φ

APPROACH T = % R = % L = % BUS STOP

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV †	REMARKS Φ

APPROACH T = % R = % L = % BUS STOP

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV †	REMARKS Φ

* DESIGNATE EACH APPROACH BY LETTER; 1-W OR 2-W (1- OR 2-WAY); PKG., N.P. (NO PKG.); ENTER DHV'S
[illegible] ..., OR COMP. (COMPOSITE PEAK.)
Φ [illegible] LANE LENGTHS -- D_2, D_3; [illegible] -- L_1, L_3; WIDENED APPROACH LENGTHS -- D_3, D_1, ETC.

BY ________

CHECKED ________

Figure 16.—Capacity analysis worksheet form.

Solution: As a first step it is necessary to check for capacity of the left-turn on each ntersection approach as discussed in part 3 under the heading *Check for Capacity of Left Turn.* A multi-phase signal control appears ikely, for which a cycle length upwards of 80 seconds is generally required. Using chart 17-B with an assumed $C=80$ seconds, the design capacity of the left-turn movement, if operating simultaneously with the opposing through movement, is $C_{D3}=70+$ on each approach; possible capacity is $C_{P3}=95$. Only the left turn E—S (70 v.p.h.) can be accommodated without a separate phase or advance green.

The opposing left turns on approach S and approach N are both relatively large and require a separate signal indication. This is a logical pattern for a third phase allowing both left-turning movements to operate simultaneously, each on a left arrow designation while all other traffic is stopped. Since the left-turning movement on approach W calls for a separate signal indication and the opposing left-turning movement on approach E does not, an advance green interval is logical and will be assumed in the preliminary analysis. On this basis, the signal phasing, with the third phase in two parts, is diagramed in figure 17.

The signal time required for moving through traffic on the expressway (phase 1) is controlled by approach S, which accommodates the larger of the two movements. Using in chart 4, $W_A=24$, $T=6\%$, $R=0\%$, $L=0\%$, (both right- and left-turning movements are on separate lanes), $MP=750{,}000$ population and $V=C_D=830$, the required G/C is 0.32.

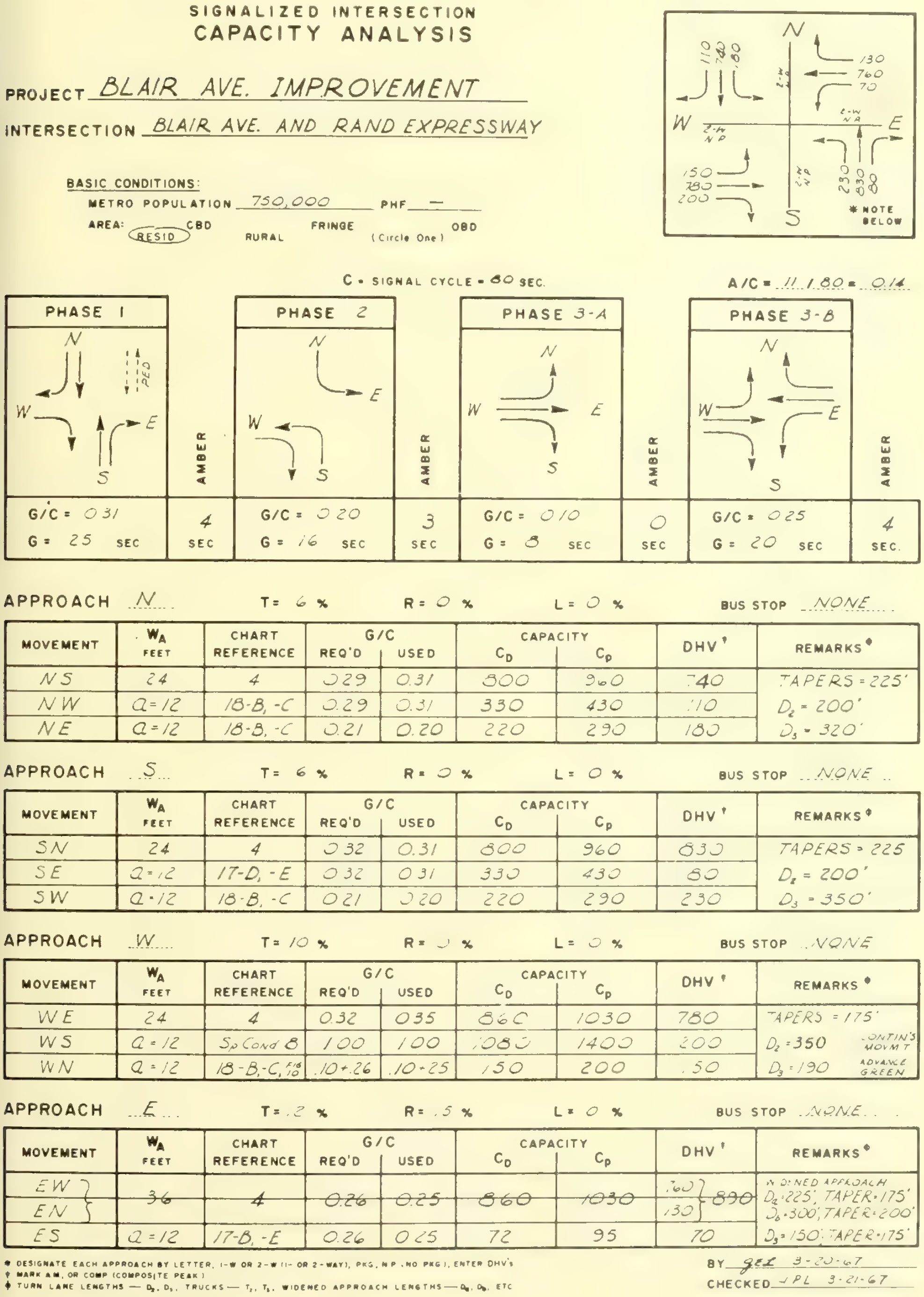
For the separate right-turn lane, on approaches S and N, using chart 17-D with $G/C=0.32$, $a=12$, $T_2=6\%$, and no pedestrian interference, the design capacity for each turning lane is found to be $C_{D2}=350$ v.p.h., which is in excess of the demand volumes. The left-turning volume of 230 v.p.h. on approach S is the controlling movement on phase 2. In chart 18-B, using $V=C_{D3}=230$ v.p.h., $T_2=6\%$, and $a=12$, the required $G'/C=0.21$.

To determine the minimum length of advance green required on approach W during phase 3, it is necessary first to determine the portion of the turning volume that can be accommodated at the end of the green period on the amber from chart 17-B; for $C=80$ seconds, it is 70 v.p.h. Volume to be accommodated by the advance green is $150-70=80$ v.p.h. Enter the nomograph in figure 10 with a volume of 80 v.p.h., and using the condition of no pedestrians, $T_3=10\%$ and $C=80$, read $G_A=8$ seconds; $G_A/C=8/80=0.10$.

The three amber periods needed for the 3-phase control are selected to be two at 4 seconds following phases 1 and 3, and one at 3 seconds following phase 2. The portion of the cycle occupied by the amber periods is $(4+4+3)/80=0.14$. The portion of the cycle remaining for the balance of phase 3, and for handling the movement on approach E, is $1.00-(0.32+0.21+0.10+0.14)=0.23$.

Total G/C for approach W during phase 3 is $0.10+0.23=0.33$. The through volume that can be discharged from this approach on two lanes at design capacity, using chart 4 with $W_A=24$, $T=10\%$, $R=0\%$, $L=0\%$, $MP=750{,}000$ population, and $G/C=0.33$, is found to be $C_D=820$ v.p.h., while the demand volume is 780 v.p.h. A continuous right-turning movement is assumed on approach W with an added lane on the approach as well as on the exit. The design capacity, as discussed in part 3 under the heading *Right-Turning Movement—Continuous, Controlled by Yield Sign, or Permitted on Red After Stop* is $1{,}200\div(1+0.10)=1{,}080$ v.p.h. This is more than adequate since the demand volume is 200 v.p.h.

The G/C available for approach E, as previously discussed, is 0.23. Testing two lanes in chart 4 for the through-plus-right volume of $V=C_D=760+130=890$ v.p.h., $W_A=24$, $T=12\%$, $R=130/890=15\%$, $L=0\%$ and $MP=750{,}000$ population, reveals the need for G/C of 0.38. To overcome the deficiency a greater width should be provided. Assuming a widened approach through the intersection, and using


N
110
740
80
130
760
70
2-W NP
2-W NA
W
E
2-W NP
150
780
200
2-W NP
230
830
80
S
* NOTE BELOW
PHASE 1
N
PED
W
E
S
AMBER
PHASE 2
N
E
W
S
AMBER
PHASE 3-A
N
W
E
S
AMBER
PHASE 3-B
N
W
E
S
AMBER

SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT BLAIR AVE. IMPROVEMENT

INTERSECTION BLAIR AVE. AND RAND EXPRESSWAY

BASIC CONDITIONS:

METRO POPULATION 750,000 PHF —

AREA: [✓ RESID] CBD RURAL FRINGE OBD (Circle One)

C = SIGNAL CYCLE = 80 SEC.

A/C = 11 / 80 = 0.14

PHASE 1	AMBER	PHASE 2	AMBER	PHASE 3-A	AMBER	PHASE 3-B	AMBER
G/C = 0.31 G = 25 SEC	4 SEC	G/C = 0.20 G = 16 SEC	3 SEC	G/C = 0.10 G = 8 SEC	0 SEC	G/C = 0.25 G = 20 SEC	4 SEC.

APPROACH N T= 6 % R= 0 % L= 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV †	REMARKS ♦
NS	24	4	0.29	0.31	800	960	740	TAPERS = 225'
NW	a=12	18-B, -C	0.29	0.31	330	430	110	D_2 = 200'
NE	a=12	18-B, -C	0.21	0.20	220	290	180	D_3 = 320'

APPROACH S T= 6 % R= 0 % L= 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV †	REMARKS ♦
SN	24	4	0.32	0.31	800	960	830	TAPERS = 225
SE	a=12	17-D, -E	0.32	0.31	330	430	80	D_2 = 200'
SW	a=12	18-B, -C	0.21	0.20	220	290	230	D_3 = 350'

APPROACH W T= 10 % R= 0 % L= 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV †	REMARKS ♦
WE	24	4	0.32	0.35	860	1030	780	TAPERS = 175'
WS	a=12	Sp Cond 8	1.00	1.00	1080	1400	200	D_2 = 350 CONTIN'S MOVMT
WN	a=12	18-B, -C, FIG 10	.10+.26	.10+.25	150	200	150	D_3 = 190 ADVANCE GREEN

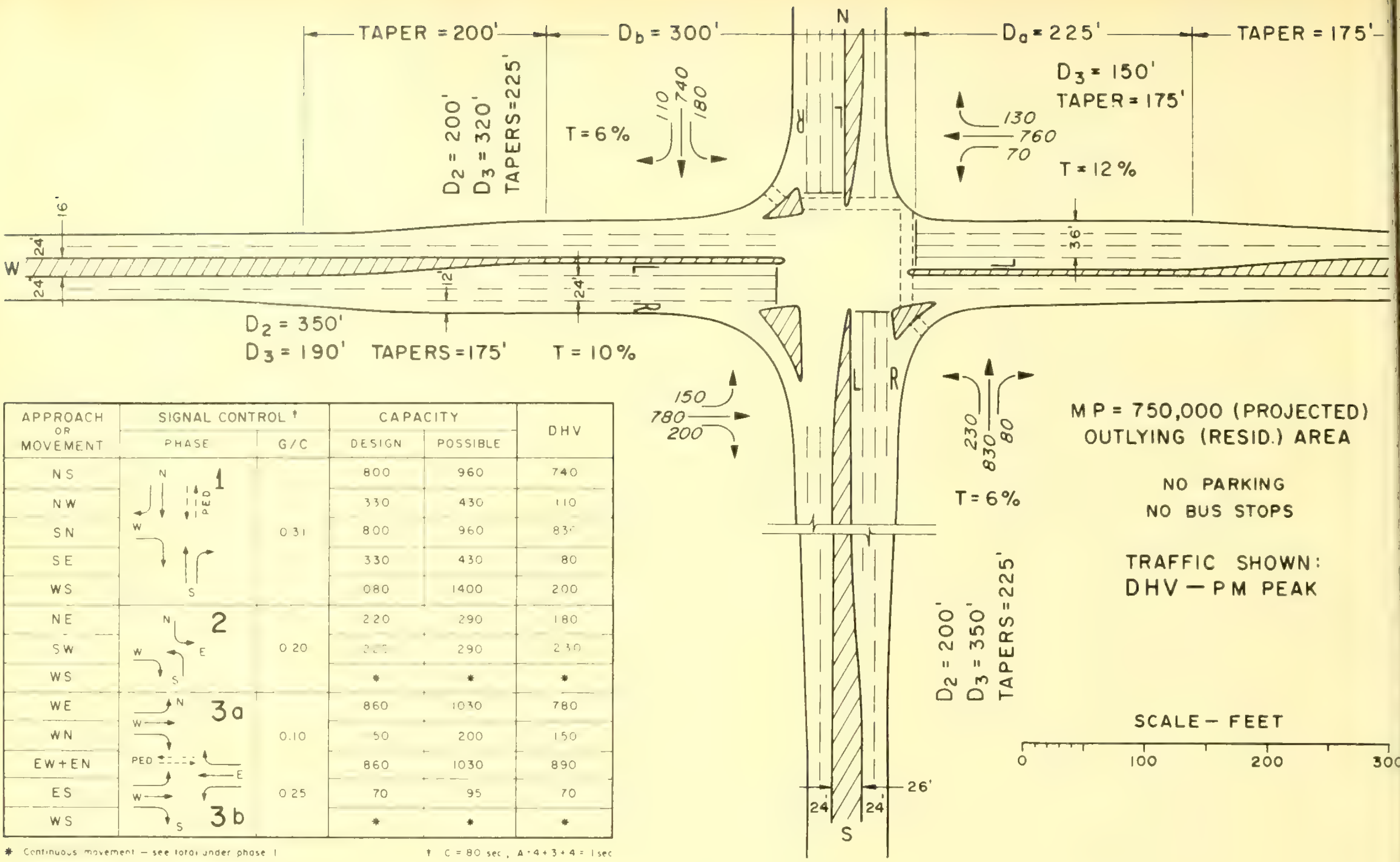
APPROACH E T= 12 % R= 15 % L= 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV †	REMARKS ♦
EW EN	~~36~~	~~4~~	~~0.26~~	~~0.25~~	~~860~~	~~1030~~	760 130 ~~890~~	WIDENED APPROACH D_a = 225', TAPER = 175' D_b = 300', TAPER = 200'
ES	a=12	17-B, -E	0.26	0.25	72	95	70	D_3 = 150', TAPER = 175'

* DESIGNATE EACH APPROACH BY LETTER, 1-W OR 2-W (1- OR 2-WAY), PKG, NP (NO PKG), ENTER DHV's
† MARK A.M., OR COMP (COMPOSITE PEAK)
♦ TURN LANE LENGTHS — D_2, D_3, TRUCKS — T_2, T_3, WIDENED APPROACH LENGTHS — D_a, D_b, ETC

BY JEL 3-20-67
CHECKED JPL 3-21-67

Figure 17.—Capacity analysis worksheet for problem 35.



APPROACH OR MOVEMENT	SIGNAL CONTROL †		CAPACITY		DHV
	PHASE	G/C	DESIGN	POSSIBLE	
NS	1	0.31	800	960	740
NW			330	430	110
SN			800	960	830
SE			330	430	80
WS			080	1400	200
NE	2	0.20	220	290	180
SW			220	290	230
WS			*	*	*
WE	3a	0.10	860	1030	780
WN			50	200	150
EW+EN	3b	0.25	860	1030	890
ES			70	95	70
WS			*	*	*

* Continuous movement — see total under phase 1

† C = 80 sec, A = 4 + 3 + 4 = 11 sec

Figure 18.—Solution for problem 35.

in chart 4 $W_A = 36$ and the other conditions noted above, find $G/C = 0.27$. The 3-lane approach although somewhat deficient for level of service C operation is considered acceptable with a slight readjustment in the other phases. Lengths of widening, as discussed in part 3 under the heading *Widened Approaches*, are $D_a = 275$ feet (minimum) preceded by 175-foot taper and $D_b = 300$ feet followed by 200-foot taper. The required lengths of turning lanes are as follows:

Approach N.—Left-turn lane, $D_3 = 320$ feet (chart 18-C) is based on the premise that, because through traffic and left-turning traffic operate on separate phases, the length of left-turn lane must be sufficient to allow vehicles to accumulate in the lane without being blocked by stored vehicles in through lanes. The controlling value is the through traffic of 740/2 = 370 v.p.h. per lane storing on the approach, which stipulates a minimum length of 320 feet in chart 18-C. Right-turn lane, $D_2 = 200$ feet, is specified for deceleration from 50 m.p.h. (figure 9). Also, based on this speed, a taper length of 225 feet is indicated preceding the turning lanes.

Approach S.—Left-turn lane, $D_3 = 350$ feet (chart 18-C), is based on the minimum storage per lane of 830/2 = 415 v.p.h. in the through lanes to allow the left-turning vehicles to clear the end of the through traffic queue. Right-turn lane, $D_2 = 200$ feet, is based on deceleration from 50 m.p.h. In addition, a taper length of 225 feet is indicated for each lane.

Approach W.—Left-turn lane, $D_3 = 190$ feet (chart 18-C), is based on storage and is larger than the dimension indicated in figure 9 for deceleration from 40 m.p.h. The left turn does not operate on a signal phase separate from the through movement on the approach, and therefore there is no need to lengthen the left-turn lane to clear the end of through traffic storage. Since the right-turn lane is designated for continuous operation, it must be long enough to clear the through traffic queue for which $D_2 = 350$ feet. This is determined in chart 18-C on the basis of a through-volume storage per lane of 780/2 = 390 v.p.h. In addition, a taper length of 175 feet (figure 9) is indicated for each turning lane.

Approach E.—Left-turn lane, $D_3 = 150$ fee (figure 9), is based on deceleration from 4 m.p.h., which is greater than the lengt required for storage in chart 18-C. A tape length of 175 feet is indicated.

A form that can be used as a capacity analysis worksheet is shown in figure 16. Th form, which can be duplicated for use i actual capacity problems, is designed t facilitate the analysis of complete intersections and to serve as a compact record o calculations. It can also be used as a companion sheet to the solution format indicate in figures 18, 22, and 26. For most intersections the entire solution can be accomplished o one copy of the form, which allows for signal phases and for 4 intersection approaches For analysis of more complex intersections two or more forms can be used. If necessary any number of trial solutions can be attempte and the computations for the workable plan retained for the record to show the preferre or selected plan.

The analysis for problem 35, althoug covered step by step in preceding discussion s also presented on the calculation form in igure 17. The simplicity and compactness of he analysis is well illustrated with all the ieeded information shown on one form. By showing the design capacity, possible capacity, and the *DHV* for each basic movement, a horough insight is gained as to the effectiveness of the solution. A design can rarely be achieved where each movement operates at precisely the desired level of service. Some of he controlling movements are set to operate at design capacity, whereas other movements esult in operation where the demand volume s either below or slightly above design apacity. Since this information is tabulated or individual movements, it also allows for comparison and weighting of results between he various movements. Thus, the analyst s able to make adjustments readily in geomery or signal timing to produce an effective overall solution and a balanced design. The analysis information is then transferred to the solution format shown in figure 18, which completely summarizes the results, including geometric requirements, and serves as a standard document for preliminary or functional design.

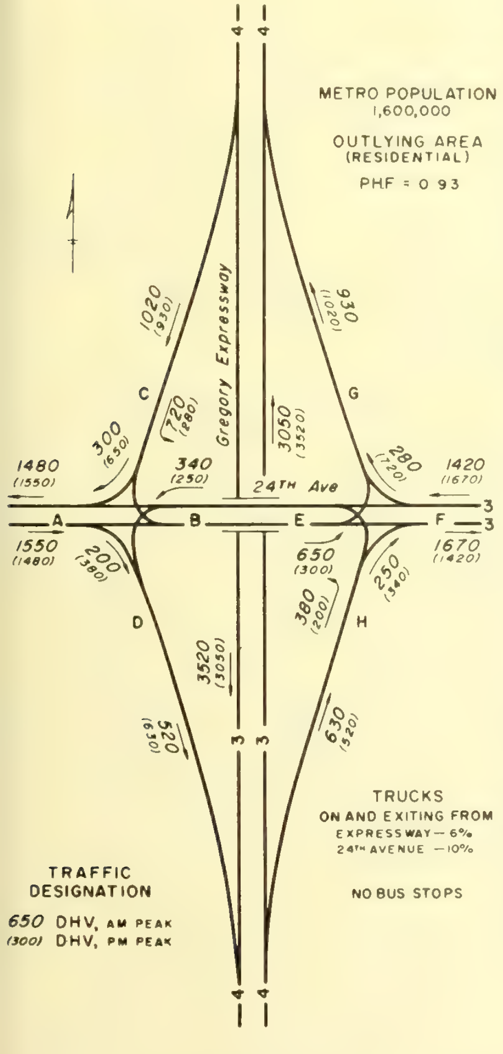
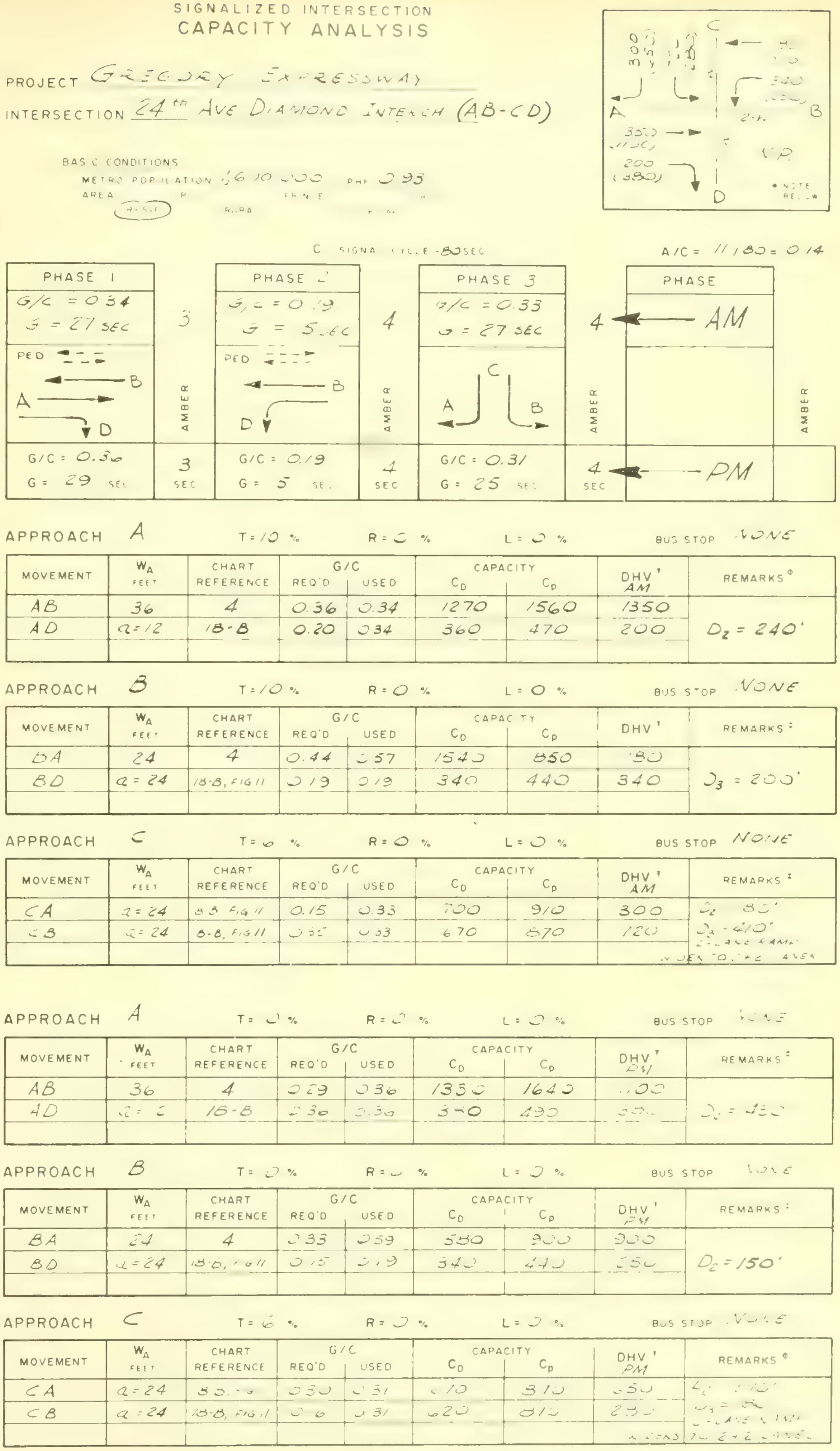


Figure 19.—Problem 36 illustrated.


SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT GREGORY EXPRESSWAY
INTERSECTION 24th AVE DIAMOND INTERCH (AB-CD)

BASIC CONDITIONS
METRO POPULATION 1,600,000 PHF 0.93
AREA
(RESID.) RURAL

C
A
B
D
300 (650)
340
2-H
350 (1150)
200 (380)
* NOTE BELOW

SIGNAL CYCLE - 80 SEC
A/C = 11/80 = 0.14

PHASE 1		PHASE 2		PHASE 3		PHASE	
G/C = 0.34 G = 27 SEC	3	G/C = 0.19 G = 5 SEC	4	G/C = 0.33 G = 27 SEC	4	AM	
PED	AMBER	PED	AMBER		AMBER		AMBER
G/C = 0.36 G = 29 SEC	3 SEC	G/C = 0.19 G = 5 SEC	4 SEC	G/C = 0.31 G = 25 SEC	4 SEC	PM	

APPROACH A T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV AM	REMARKS
AB	36	4	0.36	0.34	1270	1560	1350	
AD	a = 12	18-B	0.20	0.34	360	470	200	D_2 = 240'

APPROACH B T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV	REMARKS
BA	24	4	0.44	0.57	1540	850	180	
BD	a = 24	18-B, FIG 11	0.19	0.19	340	440	340	D_3 = 200'

APPROACH C T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV AM	REMARKS
CA	a = 24	8-B FIG 11	0.15	0.33	700	910	300	D_2 80'
CB	a = 24	8-B, FIG 11	0.35	0.33	670	870	120	D_3 - 410' 2-LANE RAMP
								WIDEN TO 2+2 LANES

APPROACH A T = 0 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV PM	REMARKS
AB	36	4	0.29	0.36	1350	1640	1100	
AD	a = 12	18-B	0.36	0.36	340	490	380	D_2 = 450'

APPROACH B T = 0 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV PM	REMARKS
BA	24	4	0.33	0.59	580	900	900	
BD	a = 24	18-B, FIG 11	0.15	0.19	340	440	250	D_2 = 150'

APPROACH C T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV PM	REMARKS
CA	a = 24	8-B, FIG 11	0.30	0.31	610	810	650	D_2 [illegible]
CB	a = 24	18-B, FIG 11	0.06	0.31	620	810	280	D_3 = [illegible] 2-LANE RAMP
								WIDEN TO 2+2 LANES

Figure 20.—Capacity analysis worksheet for problem 36, intersection AB–CD.

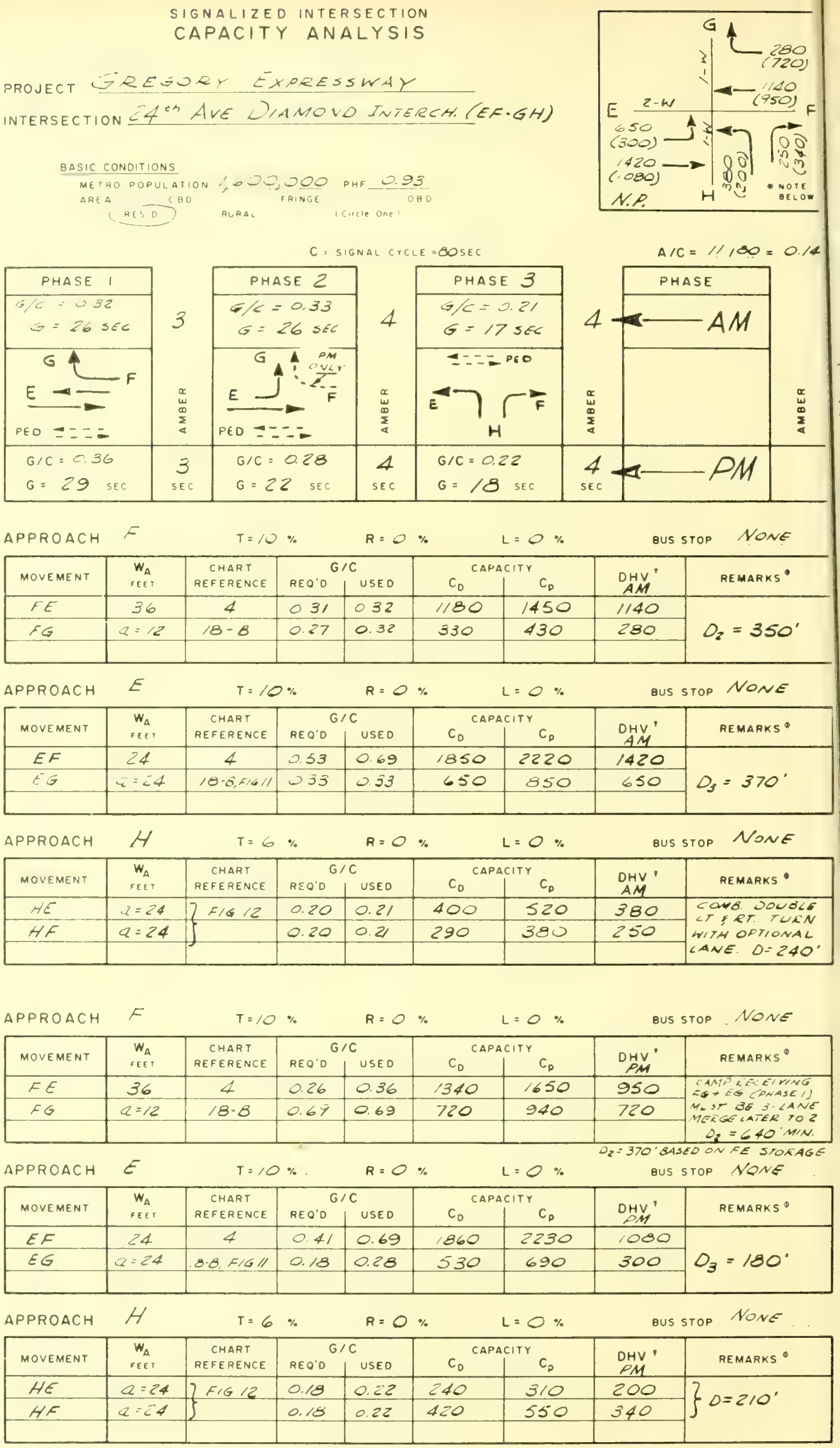
Problem 36

A diamond interchange is proposed at the crossing of a major street and an expressway in the outlying residential area of a city having a projected metropolitan population of 1.6 million. For the conditions indicated in figure 19, determine the essential geometric features of the cross street (24th Avenue) and the adjoining ramp terminals, including the number and arrangement of lanes, channelization, and signal phasing. Prepare a design sketch setting out the geometric requirements.

Solution: A detailed description of the procedural steps through the charts is not included for this problem. Instead, the solution is provided directly on the capacity analysis forms with primary references to charts 4 and 18. The results are tabulated on the worksheets in figures 20 and 21, covering intersections AB-CD and EF-GH, respectively. Both a.m. and p.m. peak-hour periods have been analyzed, and the different requirements for each are shown. The need for a 3-dial control system is apparent to allow for full efficiency and flexibility of operation to fit the characteristics and separate demands of the morning-, evening-, and off-peak periods.

An early step in the analysis of a complete intersection is the establishment of signal phasing. Sometimes there are several ways of phasing an intersection. Each way should be tested and the most efficient phasing established through preliminary use of the charts. After a phasing arrangement has been selected, the analysis is continued in a straightforward manner by determining the *G/C* required for each intersection approach based on the approach width, demand volume, and other pertinent conditions. The sum of the *G/C*'s, together with the amber periods divided by the cycle time, *A/C*, should be equal to or less than 1.00. If the *G/C*'s total less than 1.00, each *G/C* is adjusted upward by inspection or in proportion to demand on the approaches to the required total. If the *G/C*'s total more than 1.00, the design capacity or the service volume for a given level of service has been exceeded, and it may be necessary to re-analyze some of the approaches by increasing the width or changing some other condition in order to reduce the *G/C* total to approximately 1.00.

It is not always feasible to have each individual movement accommodated precisely at design capacity; this kind of balance is practically impossible. Some movements will operate at a volume below the available design capacity; other movements may operate at a volume exceeding the design capacity. Although an attempt is made to avoid the latter situation, a slight excess of demand volume over design capacity, that is, a nominal lowering of the selected level of service, is frequently acceptable as illustrated in movement AB for the a.m. peak shown in figure 20. Here the required *G/C* of 0.36 was adjusted downward to 0.34 to achieve a balance in the total *G/C;* otherwise, it would have been necessary to increase the approach from 3 to 4 lanes.


G
280
(720)
1-W
1140
(950)
E
2-W
F
650
(300)
1420
(1080)
1-W
380
(200)
250
(340)
* NOTE BELOW
N.P.
H
G
F
E
PED
G
PM ONLY
E
F
PED
PED
E
F
H

SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT GREGORY EXPRESSWAY
INTERSECTION 24th AVE DIAMOND INTERCH. (EF·GH)

BASIC CONDITIONS
METRO POPULATION 1,600,000 PHF 0.93
AREA CBD FRINGE OBD [✓ RES D] RURAL (Circle One)

C = SIGNAL CYCLE = 80 SEC

A/C = 11/80 = 0.14

PHASE 1	AMBER	PHASE 2	AMBER	PHASE 3	AMBER	PHASE	AMBER
G/C = 0.32 G = 26 SEC	3	G/C = 0.33 G = 26 SEC	4	G/C = 0.21 G = 17 SEC	4	AM	
G/C = 0.36 G = 29 SEC	3 SEC	G/C = 0.28 G = 22 SEC	4 SEC	G/C = 0.22 G = 18 SEC	4 SEC	PM	

APPROACH F T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* AM	REMARKS*
FE	36	4	0.31	0.32	1180	1450	1140	D_2 = 350'
FG	a = 12	18-8	0.27	0.32	330	430	280	

APPROACH E T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* AM	REMARKS*
EF	24	4	0.53	0.69	1850	2220	1420	D_3 = 370'
EG	a = 24	18-6, FIG 11	0.33	0.33	650	850	650	

APPROACH H T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* AM	REMARKS*
HE	a = 24	FIG 12	0.20	0.21	400	520	380	COMB. DOUBLE LT & RT. TURN WITH OPTIONAL LANE. D = 240'
HF	a = 24		0.20	0.21	290	380	250	

APPROACH F T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* PM	REMARKS*
FE	36	4	0.26	0.36	1340	1650	950	RAMP RECEIVING FG + EG (PHASE 1) MUST BE 3-LANE MERGE LATER TO 2 D_2 = 640' MIN.
FG	a = 12	18-8	0.69	0.69	720	940	720	

D_2 = 370' BASED ON FE STORAGE

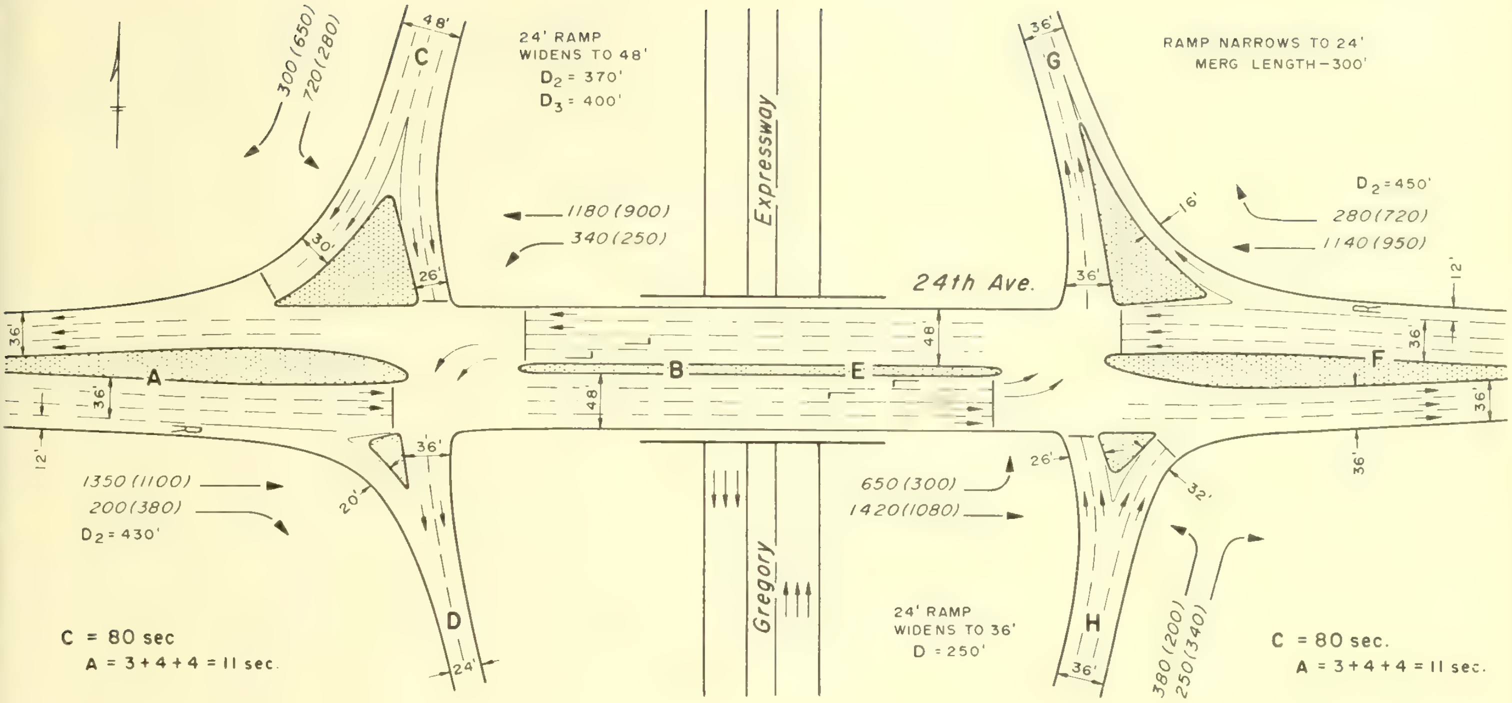
APPROACH E T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* PM	REMARKS*
EF	24	4	0.41	0.69	1860	2230	1080	D_3 = 180'
EG	a = 24	18-8, FIG 11	0.18	0.28	530	690	300	

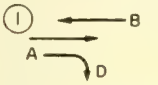
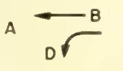
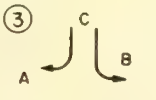
APPROACH H T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_p	DHV* PM	REMARKS*
HE	a = 24	FIG 12	0.18	0.22	240	310	200	D = 210'
HF	a = 24		0.18	0.22	420	550	340	

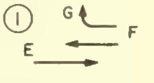
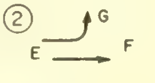
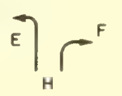
Figure 21.—Capacity anlaysis worksheet for problem 36, intersection EF–GH.



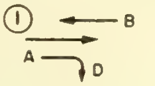
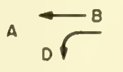
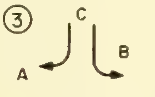
INTERSECTION AB–CD AM PEAK

APPROACH OR MOVEMENT	SIGNAL CONTROL		CAPACITY		DHV
	PHASE	G/C	DESIGN	POSSIBLE	
AB	① 	0.34	1270	1560	1350
AD			360	470	200
BA	② 	0.19	1540	1850	1180
BD			340	440	340
CA	③ 	0.33	700	910	300
CB			670	870	720

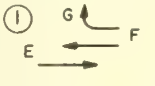
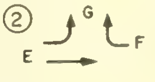
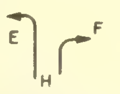
INTERSECTION DE–FG AM PEAK

APPROACH OR MOVEMENT	SIGNAL CONTROL		CAPACITY		DHV
	PHASE	G/C	DESIGN	POSSIBLE	
FE	① 	0.32	1180	1450	1140
FG			330	430	280
EF	② 	0.33	1850	2220	1420
EG			650	850	650
HE	③ 	0.21	400	520	380
HF			290	380	250

INTERSECTION AB–CD PM PEAK

AB	① 	0.36	1330	1640	1100
AD			380	490	380
BA	② 	0.19	1580	1900	900
BD			340	440	250
CA	③ 	0.31	670	870	650
CB			620	810	280

INTERSECTION DE–FG PM PEAK

FE	① 	0.36	1340	1650	950
FG ✱			720	940	720
EF	② 	0.28	1860	2230	1080
EG			530	690	300
FG ✱			✱	✱	✱
HE	③ 	0.22	240	310	200
HF			420	550	340

✱ Moves during phases 1 and 2 (G/C = 0.69); total shown under phase 1

Figure 22.—Solution for problem 36.

The solution, showing the design requirements, is indicated in figure 22. The tabulations below the geometric layout are part of the suggested format for summarizing the analysis. Both a.m. and p.m. peaks are considered, and the selected signal phasing and a comparison for each movement of the *DHV* with design capacity and possible capacity are shown. The other information tabulated on the worksheets of figures 20 and 21—such as the number, arrangement, and lengths of lanes—is reflected in the design sketch of figure 22. Several features developed in this design are characteristic of diamond interchanges in urban areas. The cross street widens within the interchange to accommodate left-turning vehicles. In urban areas, the 2-abreast left-turn design on the cross street is sometimes required, frequently for one and occasionally for both left-turning movements. The 4-lane approach at C divides into a 2-lane right-turning movement and a 2-lane left-turning movement to handle the indicated volumes. The ramp proper widens from a 2-lane width at the expressway exit to a 4-lane section at C. Lengths called for on the widened portion, D_2 and D_3, are indicated on the plan. The requirement on approach H is a 3-lane section, separating into two 2-lane turning movements with the center lane serving as an optional lane. Exit G is designed for 3 lanes to allow the 2-lane movement EG to merge with the relatively large movement FG during phase 2 in the evening peak. The ramp then narrows to 2 lanes before entering the expressway.

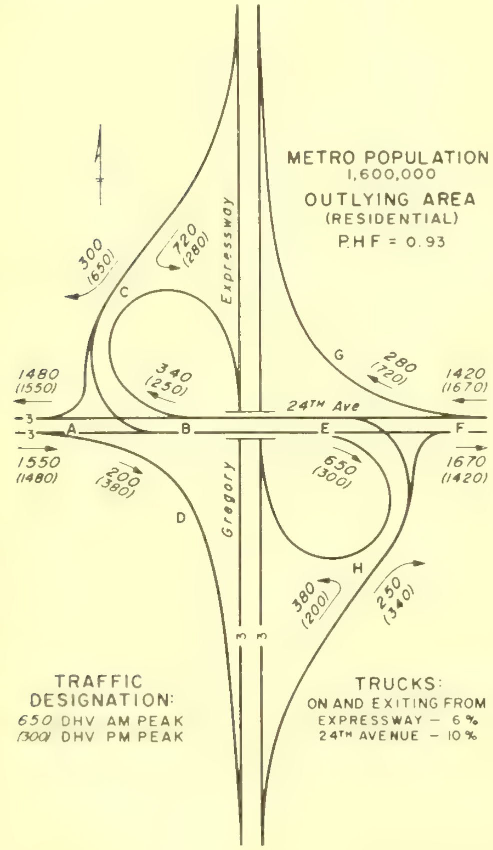
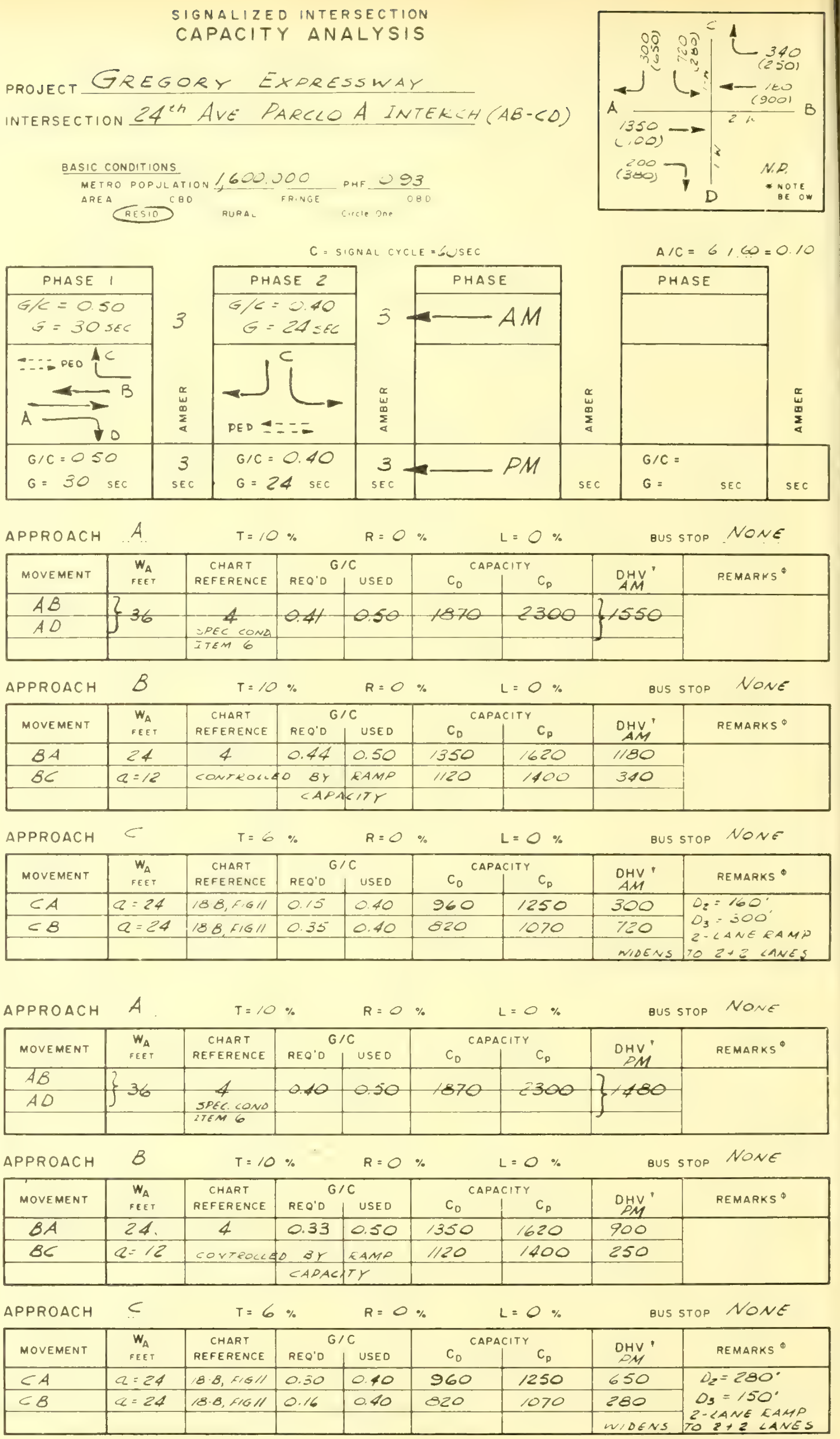


Figure 23.—Problem 37 illustrated.



SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT GREGORY EXPRESSWAY

INTERSECTION 24th AVE PARCLO A INTERCH (AB-CD)

BASIC CONDITIONS

METRO POPULATION 1,600,000 PHF 0.93

AREA CBD FRINGE OBD [✓ RESID] RURAL Circle One

C
A
B
D
300 (650)
720 (280)
340 (250)
160 (900)
1350 (100)
200 (380)
2 L
N.P.
* NOTE BELOW
PED
AM
PM

C = SIGNAL CYCLE = 60 SEC

A/C = 6 / 60 = 0.10

PHASE 1		PHASE 2		PHASE		PHASE	
G/C = 0.50 G = 30 SEC	3	G/C = 0.40 G = 24 SEC	3	AM			
	AMBER		AMBER		AMBER		AMBER
G/C = 0.50 G = 30 SEC	3 SEC	G/C = 0.40 G = 24 SEC	3 SEC	PM	SEC	G/C = G = SEC	SEC

APPROACH A T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* AM	REMARKS*
AB	} 36	4	0.41	0.50	1870	2300	} 1550	
AD		SPEC COND ITEM 6						

APPROACH B T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* AM	REMARKS*
BA	24	4	0.44	0.50	1350	1620	1180	
BC	a = 12	CONTROLLED BY RAMP CAPACITY			1120	1400	340	

APPROACH C T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* AM	REMARKS*
CA	a = 24	18 B, FIG 11	0.15	0.40	960	1250	300	D_2 = 160' D_3 = 300'
CB	a = 24	18 B, FIG 11	0.35	0.40	820	1070	720	2-LANE RAMP
							WIDENS	TO 2 + 2 LANES

APPROACH A T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* PM	REMARKS*
AB	} 36	4	0.40	0.50	1870	2300	} 1480	
AD		SPEC. COND ITEM 6						

APPROACH B T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* PM	REMARKS*
BA	24	4	0.33	0.50	1350	1620	900	
BC	a = 12	CONTROLLED BY RAMP CAPACITY			1120	1400	250	

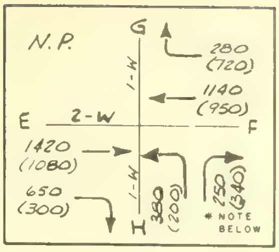
APPROACH C T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C REQ'D	G/C USED	CAPACITY C_D	CAPACITY C_P	DHV* PM	REMARKS*
CA	a = 24	18-B, FIG 11	0.30	0.40	960	1250	650	D_2 = 280'
CB	a = 24	18-B, FIG 11	0.16	0.40	820	1070	280	D_3 = 150' 2-LANE RAMP
							WIDENS	TO 2 + 2 LANES

Figure 24.—Capacity analysis worksheet for problem 37, intersection AB–CD.

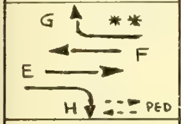
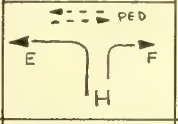
SIGNALIZED INTERSECTION
CAPACITY ANALYSIS

PROJECT GREGORY EXPRESSWAY
INTERSECTION 24th AVE PARCLO A INTERCH. (EF-GH)



BASIC CONDITIONS
METRO POPULATION 1,600,000 PHF 0.93
AREA CBD FRINGE OBD
[✓ RESID] RURAL (Circle One)

C = SIGNAL CYCLE = 60 SEC
A/C = 6 / 60 = 0.10

PHASE 1		PHASE 2		PHASE		PHASE	
G/C = 0.65 G = 39 SEC	3	G/C = 0.25 G = 15 SEC.	3	← AM		** NOTE: LAGGING GREEN ON FG FOR 3 SEC DURING PM PEAK ONLY. (G/C = 0.70)	
	AMBER		AMBER		AMBER		AMBER
G/C = 0.65 G = 39 SEC	3 SEC	G/C = 0.25 G = 15 SEC	3 SEC	← PM	SEC	G/C = G = SEC	SEC

APPROACH F T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	AM	
FE	24	4	0.42	0.65	1750	2100	1140	D_2 = 240'
FG	a = 12	18-8	0.27	0.65	670	870	280	

APPROACH E T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	AM	
EF	24	4	0.53	0.65	1750	2100	1420	
EH	a = 12	CONTROLLED	BY	RAMP	1120	1400	650	
			CAPACITY					

APPROACH H T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	AM	
HE	a = 24	} FIG 12	0.20	0.25	480	620	380	COMB. DOUBLE LT. & RT. TURN WITH OPTIONAL LANE. D = 190'
HF	a = 24		0.20	0.25	310	400	250	

APPROACH F T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	PM	
FE	24	4	0.36	0.65	1750	2100	950	D_2 = 470' DES. OR D_2 = 420 MIN.
FG	a = 12	18-8	0.69	0.70	730	950	720	
							BASED	ON FE STORAGE

APPROACH E T = 10 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	PM	
EF	24	4	0.41	0.65	1750	2100	1080	
EH	a = 12	CONTROLLED	BY	RAMP	1120	870	300	
			CAPACITY					

APPROACH H T = 6 % R = 0 % L = 0 % BUS STOP NONE

MOVEMENT	W_A FEET	CHART REFERENCE	G/C		CAPACITY		DHV †	REMARKS Φ
			REQ'D	USED	C_D	C_P	PM	
HE	a = 24	} FIG 12	0.18	0.25	280	360	200	} D = 180'
HF	a = 24.		0.18	0.25	470	610	340	

Figure 25.—Capacity anlysis worksheet for problem 37, intersection EF–GH.

The capacity analysis, using the 3-phase control at intersection AB–CD and at intersection EF–GH, provides the basis for the geometric design of the interchange along 24th Avenue. A more refined and complete analysis of signalization, including the use of time-space diagrams and overlap intervals to provide maximum degree of coordination between the two intersections and progression of movements, also can be an important aspect of design. Usually, however, this is not essential in establishing the basic geometry of the intersection. Sometimes a nominal increase in capacity is achieved through such refinement; but, if not accounted for initially, it places the design on the safe side.

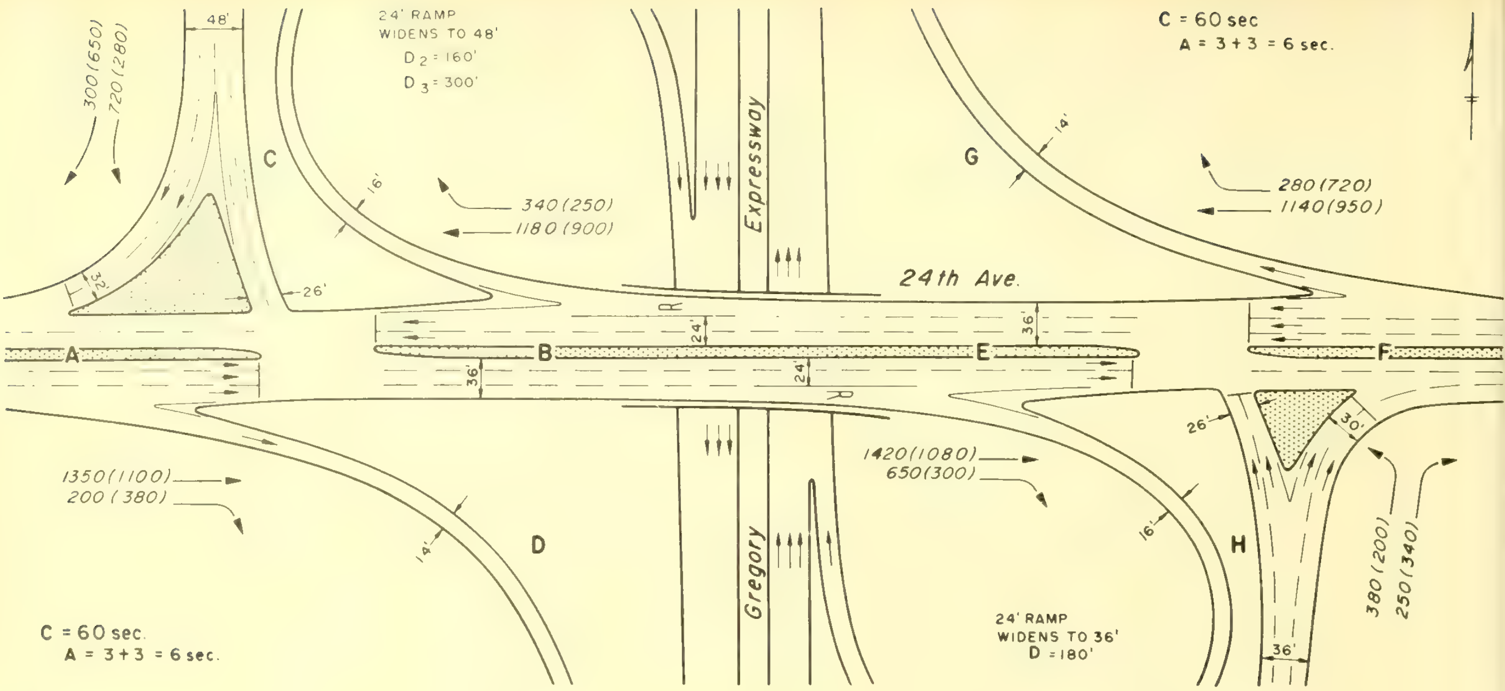
Problem 37

For the basic conditions described in problem 36, a parclo-A interchange of the form shown in figure 23 is also to be considered at the same location. As before, determine the essential geometric features of the cross street and the adjoining ramp terminals, including the number and arrangement of lanes, channelization, and signal phasing. Prepare a design sketch setting forth the geometric requirements.

Solution: The analysis and procedure in the solution are much the same as in the previous problem. An important initial step in the analysis is the determination of signal phasing. For a parclo-A a simple 2-phase control at each intersection is all that is required. Complete analysis for intersections AB–CD and EF–GH for both a.m. and p.m. peaks are detailed on the worksheets of figures 24 and 25. The solution, including the geometric layout, signal timing, and summary of volume-capacity relations, is shown in figure 26.

Basically, the parclo-A requires no widening of the cross street through the interchange. Moreover, the space occupied by the approaches to the interchange is equivalent to the normal street width. The designs for the movements exiting from the cross street differ significantly from the diamond interchange; however, the designs for the entering ramp terminals, approaches C and H, are essentially the same as on the diamond interchange. In this example, the parclo-A requires not only a lesser number of lanes than the diamond to handle the same traffic, but operates at an overall higher level of service; that is, for the majority of the movements there is a greater difference between capacity and *DHV*.

To achieve a more meaningful comparison of the two alternative plans with regard to capacity potential, a *design capacity index* can be employed. This measure of performance or overall ability of the intersection to handle traffic is the ratio of the sum of all traffic entering the intersection, during a given period of time, to the sum of all approach design capacities handling this traffic. In brief, it is the composite V/C_D, representing the a.m. peak, the p.m. peak, or a combination of the two. For example, the design capacity index, V/C_D, for intersection AB–CD during



INTERSECTION AB-CD AM PEAK

APPROACH OR MOVEMENT	SIGNAL CONTROL		CAPACITY		DHV
	PHASE	G/C	DESIGN	POSSIBLE	
	① C				
AB+AD	PED B	0.50	1870	2300	1550
BA	A		1350	1620	1180
BC	D		1120	1400	340
CA	② C		960	1250	300
CB	A B	0.40	820	1070	720
	PED				

INTERSECTION EF-GH AM PEAK

APPROACH OR MOVEMENT	SIGNAL CONTROL		CAPACITY		DHV
	PHASE	G/C	DESIGN	POSSIBLE	
FE	① G		1750	2100	1140
FG	F	0.65	670	870	280
EF	E		1750	2100	1420
EH	H PED		1120	1400	650
HE	② PED		480	620	380
HF	E F	0.25	310	400	250
	H				

INTERSECTION AB-CD PM PEAK

	① C				
AB+AD	PED B		1870	2300	1480
BA	A	0.50	1350	1620	900
BC	D		1120	1400	250
CA	② C		960	1250	650
CB	A B	0.40	820	1070	280
	PED				

INTERSECTION EF-GH PM PEAK

FE	① G		1750	2100	950
FG ✱	F		730	950	720
EF	E	0.65	1750	2100	1080
EH	H PED		1120	870	300
HE	② PED		280	360	200
HF	E F	0.25	470	610	340
	H				

✱ FG has 3-sec. lagging green during PM peak only, yielding G/C = 0.70

Figure 26.—Solution for problem 37.

the a.m. peak is: on the diamond interchange, 4,090/4,880=0.84; on the parclo-A interchange, 4,090/6,020=0.68. The indices indicate that, as a whole, the diamond is operating at 84 percent of design capacity, and the parclo-A at 68 percent of design capacity. Comparison of the two operating percentages in itself is not fully indicative of conditions. It is also necessary to consider the number of individual movements or vehicles operating at V/C_D larger than 1.00. For the diamond interchange, movements AB and CB are in this category; but on the parclo-A interchange, all movements are well below the ratio of 1.00.

Another more detailed way of evaluating and comparing volume-capacity relations on alternative designs is by determining design capacity indices along with levels of service for individual movements. The following tabulation, compiled from the summary tables on figures 22 and 26, allows for a comparison of the a.m. peak of intersection AB–CD on the diamond with the a.m. peak of intersection AB–CD on the parclo-A.

Diamond Interchange

Movement	V	C_D	V/C_D	*Level of Service*
AB	1,350	1,270	1.06	D
AD	200	360	0.56	A
BA	1,180	1,540	0.77	A
BD	340	340	1.00	C
CA	300	700	0.43	A
CB	720	670	1.07	D
Design Capacity Index	4,090 ÷	4,880 =	0.84	A–D

Parclo-A Interchange

Movement	V	C_D	V/C_D	*Level of Service*
AB+AD	1,550	1,870	0.83	A
BA	1,180	1,350	0.87	A
BC	340	1,120	0.30	A
CA	300	860	0.35	A
CB	720	820	0.88	A
Design Capacity Index	4,090 ÷	6,020 =	0.68	A

The levels of service shown in the tabulations were determined by comparing the V/C_D ratios with the f factors in tables 3 and 5. A thorough insight is gained with regard to operation and capacity potential of the two designs. Note the superiority of the parclo-A interchange when the levels of service of individual movements and the design capacity indices are compared. The operation of a complete intersection cannot be rated with a single level of service, unless all component movements of the intersection operate at one given level. Thus, as shown above, the intersection on the diamond is rated as a range level of service A–D, with a design capacity index of 0.84. The intersection on the parclo-A is rated—level of service A with a design capacity index of 0.68.

PART 5—OVERALL INTERSECTION CAPACITY

AS PART of planning and preliminary design processes, a quick, approximate determination of capacities is often needed. The problem usually resolves itself into one of two conditions: (1) where the approach volumes and street widths are known, the adequacy of the capacity of the intersection must be determined; or (2) where the approach volumes and the width of one street are known, the width of the intersecting street must be determined. The need for analysis may pertain to an individual intersection, or may extend to a route with a series of intersections, or possibly to a whole system of streets in a given sector of a city. Charts 19 and 20 were devised for this purpose; they combine the necessary information for both of the intersecting streets on one chart and give results in terms of overall capacity. Each chart takes into account jointly, for average conditions, the intersection of any two facilities, regardless of 1-way or 2-way operation, type of area, and parking regulation. The left half of the chart is used for the approach on one street and the right half for the approach on the other or intersecting street. A line projected between the inner sides of the two halves of the chart determines, at the intersection of the y-y axis and related metropolitan sizes, the adequacy of the street intersection.

The two parts of charts 19 and 20 are identical except for the reverse plotting. The arrangement of each part is similar to that of charts 1 and 2, but the G/C ratio in charts 19 and 20 is made the outer scale and the volume is shown as the lower series of curves. The G/C ratio on the side scales is the proportion of time required on the one approach for operation at design capacity. Assuming that 10 percent of the cycle time is being used in amber periods, design capacity is obtained when the total of two green intervals is 90 percent of the cycle (the sum of the two G/C values $=0.90$). The 1.00 point on the y-y axis is located so that a straight line between any two G/C values passes through the 1.00 point when their sum is 0.90. The scale values above and below the 1.00 point on the y-y axis show the proportion by which the sum of the G/C values is deficient or in excess of the design capacity condition.

The scale on the axis also gives V/C_D, the ratio of approach volume to design capacity, combined for the two approaches. Thus, when the combined approach volumes equal the combined design capacities, the ratio is 1.00 (level of service C operation). Points on the scale below 1.00 indicate operation at superior levels of service, B or A. Points above 1.00 indicate operation at less favorable levels of service, D, E, and F. Possible capacity, level E, is the value on the y-y axis corresponding to the average f value for the two approaches as found in table C on charts 19 and 20. Also, any reading on the axis can be compared with the average values for the two approaches in tables 3 and 4 to find the level of service at which the intersection as a whole is operating. The y-y axis is representative of conditions in metropolitan areas of 250,000 population. To allow for adjustment of results to other metropolitan sizes, bar scales parallel to the y-y axis are included.

Charts 19 and 20 are intended for preliminary design and general evaluation of operation and capacity of intersections, including analyses of a series of intersections and street systems. The charts incorporate numerous specific conditions and several average conditions. Specific conditions accounted for are approach width; 1-way or 2-way operation; parking regulation; area of city, such as *CBD*, fringe, etc.; approach volume, G/C; and metropolitan area population. Average conditions built into the charts assume 5 percent trucks, 10 percent right turns, 10 percent left turns, and no bus stops. Allowance of 10 percent total for amber periods is incorporated in the charts as a constant. Under normal circumstances the deviation from these average conditions is not significant. Moreover, those variables that generally have a pronounced effect on capacity allow for adjustment in the charts. Hence, the results produced are reasonably correct for regular intersections.

Charts 19 and 20 also may be adapted to other than regular intersections for planning and preliminary design, as follows:

(1) Approaches with separate left-turn lane, not requiring separate signal indication: Deduct 10 percent of the approach volume or 100 v.p.h., whichever is smaller, and use the width of approach exclusive of the separate left-turn lane.

(2) Approaches with separate left-turn lane, requiring separate signal indication: Deduct left-turn volume from approach volume. Use the width of approach exclusive of the separate left-turn lane. Allow an additional G/C of 0.10 for left-turning volumes of 120 to 140 v.p.h., and an additional $G/C=$ (v.p.h. turning left)÷1,000 for larger volumes. Thus, for a left-turning volume of 200 v.p.h., the additional G/C would equal 0.20. The procedure in the chart for applying the additional G/C is demonstrated in problem 40 and figure 28(1).

(3) Intercepted approaches at T or Y intersections: The capacities of intercepted approaches are lower than on the approaches

of normal (4-leg) intersections, as discussed in part 3 under the heading *T and Y Intersections*. The overall adjustment, indicated therein, for the angle of turn of the predominant movement may be applied to the solution in charts 20 and 21. For angles of turn in the vicinity of 90 degrees, capacities or service volumes on intercepted approaches are approximately 0.80 of the capacities or service volumes on 4-leg intersection approaches for the same width and traffic conditions. Thus, if the end product in chart 19 or 21 is the approach (service) volume for an intercepted approach, the result should be multiplied by 0.80. If the end product is the V/C_D ratio, the G/C indicated for the intercepted approach should be divided by 0.80, and the adjusted G/C should then be used to complete the solution.

The various applications of charts 19 and 20 are demonstrated in the following problems.

Problem 38

In the *CBD* of a 2 million population metropolitan area, a 1-way street (approach A) 42 feet wide with parking on both sides intersects a 2-way street (approach B) 66 feet wide without parking. Conditions are assumed to be average. Determine whether the intersection capacity is adequate when the peak-hour volume in one direction on approach A is 900 v.p.h. and on approach B 1,280 v.p.h.

Solution: For approach A, enter chart 19 at upper left with $W_A=42$, proceed right to the curve for 1-way street with parking on both sides in the *CBD*, then down to an approach volume of 900 v.p.h. and to the right where a $G/C=0.41$ is intersected. For approach B, enter chart at extreme right with $W_A=66/2=33$, proceed left to the curve for 2-way street without parking in the *CBD*, then down to an approach volume of 1,280 v.p.h., and to the left where a $G/C=0.56$ is intersected. Find the intersection point on the y-y axis by drawing a straight line between the two G/C values. Project horizontally to the left, intersecting the *MP* scale for over 1 million population; read $V/C_D=0.90$.

Because the result falls below the level of service C line (ratio 1.00), operation is superior to level of service C; that is, the demand volume is below design capacity. To find the required G/C at design capacity for approach A and approach B, for the indicated city size, to handle the volume of 900 and 1,280 v.p.h., respectively, the G/C values previously found should be divided by the adjustment factor shown in the chart along the top of the *MP* scales (see footnote under left part of chart). Thus, for approach A the required $G/C=0.41/1.20=0.34$, and for approach B the required $G/C=0.56/1.20=0.47$. If it is desired to adjust the G/C values proportionally to acheive balanced operation, the required G/C values should be divided by the V/C_D ratio or $G/C=0.34/0.90=0.38$ for approach A and $G/C=0.47/0.90=0.52$ for approach B; that is, including 10 percent of the cycle for amber, the total$=0.38+0.52+0.10=1.00$.

Problem 39

In the *OBD* of a 750,000 population metropolitan area, a 2-way street (approach A) 86 feet wide with parking intersects a 1-way street (approach B), 47 feet wide with parking on one side. Conditions are assumed to be average. The *DHV* on approach A is 1,200 v.p.h. and on approach B it is 2,200 v.p.h. Determine the following:

(1) Level of service at which the intersection would operate.

(2) The extent to which traffic can be increased on approach A—keeping approach B constant—to produce possible capacity operation on the intersection as a whole.

(3) The extent to which traffic can be increased uniformly in solution (1) to produce possible capacity operation on the intersection as a whole; what would be the signal timing?

(4) The required width of approach B for operation at design capacity, if parking on approach B is removed and all other conditions remain the same; what would be the signal timing?

Solution: The solutions for the four parts of this problem are illustrated by the schematics in figure 27, which show the various ways in which charts 19 and 20 can be used.

Part 1.—To find the level of service, first the V/C_D ratio must be found in chart 20. The procedure is demonstrated in figure 27(1). For approach A, course *a-b-c-d* is followed, producing on the left G/C scale an intercept of 0.44; for approach B, course *e-f-g-h* is followed, producing on the right G/C scale an intercept of 0.61; points *d* and *h* are connected by a straight line intersecting the y-y axis at point *i*; a horizontal projection from *i* to point *j* on the *MP* scale for 750,000 population yields the result $V/C_D=1.06$. Because this ratio is larger than 1.00, the demand volume exceeds design capacity or level C service volume. Demand volume, however, is well within possible capacity (level E) limitations, for which the V/C_D ratio would be 1.25, the average *f* value for the two approaches determined in table C (chart 20). To find the specific level of service at which the intersection is operating, it is necessary to consult tables 3 and 4. The composite *f* values for the two approaches, representing various levels of service, are the averages for the 43-foot approach on a 2-way street with parking and for the 47-foot approach on a 1-way street with parking on one side. These are level A, 0.87; level B, 0.90; level C, 1.00; level D, 1.15; and level E, 1.25. The V/C_D ratio of 1.06 found in chart 20 for this intersection falls between 1.00 and 1.15, the limiting values of levels C and D. The intersection, therefore, operates at level D during peak hours.

Part 2.—To find the extent to which traffic may be increased on approach A to correspond to possible capacity, the composite *f* value for approaches A and B of 1.25, as found in part 1, must be used in chart 20. The chart is entered by locating a point at 1.25 on the V/C_D scale for $MP=750{,}000$. This is point *k* as shown in the schematic of figure 27(2). Proceed to the right horizontally from *k* to intercept point *l* on y-y axis. Enter chart at the upper right for approach B, following as before course *e-f-g-h*. From *h* project a straight line through *l* to point *m* at the intersection with the left G/C scale. Proceed horizontally to the left from *m* to the intersection point *n* on the previously established course *a-b-c* for approach A. Read at intersection point *n* an approach A volume of 1,730 v.p.h.

Part 3.—To find the extent to which traffic may be increased on both approaches uniformly to obtain possible capacity operation, the chart is entered with $V/C_D=1.25$ as in part 2, establishing points *k* and *l*. As shown in the schematic of figure 27(3), draw line *pq* through point *l* parallel to line *dh*, set in the solution of part 1. Project a horizontal line to the left from point *p* to intersect point *r* on course *a-b-c* situated before; read a volume of 1,470 v.p.h. on approach A. Project a horizontal line to the right from point *q* to intersect point *s* on the previously set course *e-f-g*; read a volume of 2,500 v.p.h. on approach B. The G/C values of 0.54 and 0.70 at points *p* and *q* pertain to *MP* base of 250,000 and to

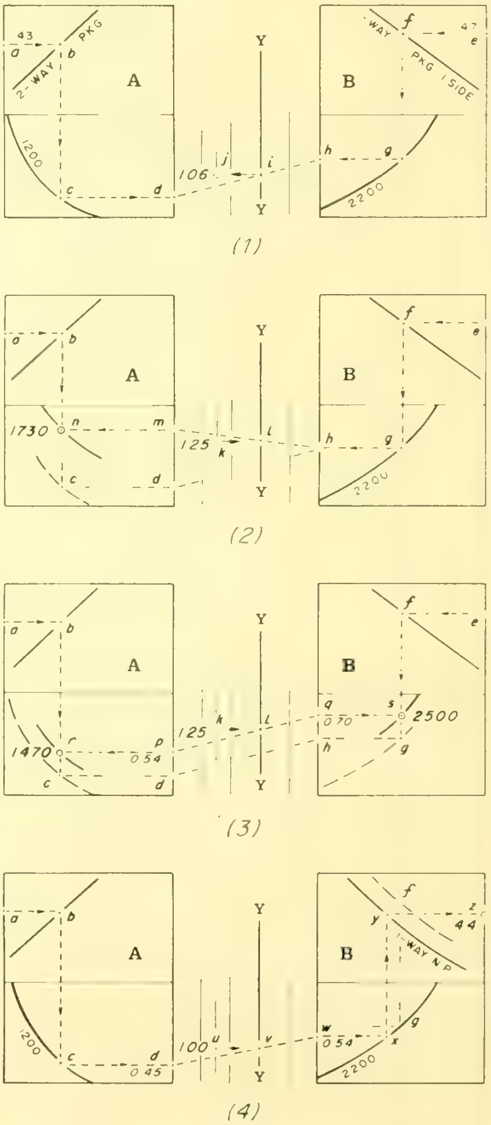


Figure 27.—Chart solutions for problem 39.

TABLE C — RATIO OF POSSIBLE CAPACITY TO DESIGN CAPACITY

STREET TYPE	PARKING CONDITIONS	FACTOR f, when W_A (in feet) is —								
		10	15	20	25	30	35	40	50	60
TWO WAY	No Parking	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
	With Parking	—	—	1.10	1.14	1.18	1.21	1.25	1.31	1.34
ONE WAY	No Parking	—	—	1.15	1.13	1.12	1.12	1.13	1.15	1.17
	Parking One Side	—	—	1.10	1.13	1.16	1.18	1.20	1.25	1.30
	Parking Both Sides	—	—	—	1.25	1.25	1.25	1.27	1.32	1.37

EXAMPLE

Given

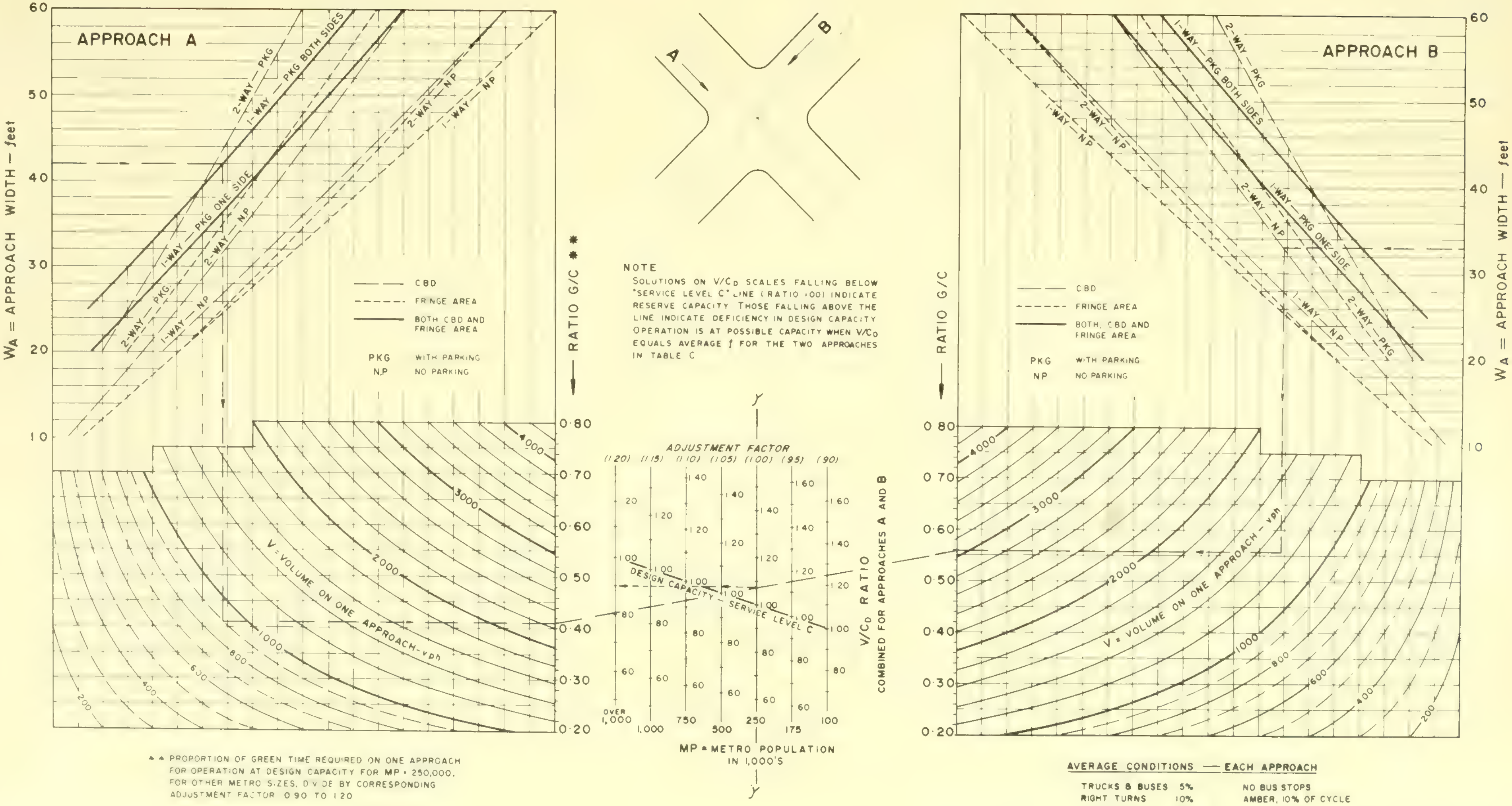
MP = 2,000,000, C.B.D.

APPROACH A:
One-way street
Parking both sides
$W_A = 42'$
DHV = 900 vph

APPROACH B:
Two-way street
No parking
$W_A = 33'$
DHV = 1280 vph

Solution

$V/C_D = 0.90$
Operation superior to service level C, i.e. demand volume below design capacity.



NOTE
SOLUTIONS ON V/C_D SCALES FALLING BELOW "SERVICE LEVEL C" LINE (RATIO 1.00) INDICATE RESERVE CAPACITY. THOSE FALLING ABOVE THE LINE INDICATE DEFICIENCY IN DESIGN CAPACITY. OPERATION IS AT POSSIBLE CAPACITY WHEN V/C_D EQUALS AVERAGE f FOR THE TWO APPROACHES IN TABLE C.

** PROPORTION OF GREEN TIME REQUIRED ON ONE APPROACH FOR OPERATION AT DESIGN CAPACITY FOR MP = 250,000. FOR OTHER METRO SIZES, DIVIDE BY CORRESPONDING ADJUSTMENT FACTOR 0.90 TO 1.20

AVERAGE CONDITIONS	— EACH APPROACH
TRUCKS & BUSES 5%	NO BUS STOPS
RIGHT TURNS 10%	AMBER, 10% OF CYCLE
LEFT TURNS 10%	

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

OVER-ALL INTERSECTION CAPACITY — AVERAGE URBAN CONDITIONS
C.B.D. AND FRINGE AREA
CHART 19

the volumes if handled at design capacity. These values, however, can be used proportionately to establish the signal timing for possible capacity operation, as follows: approach A, $G/C=0.54/(0.54+0.70)=0.39$; approach B, $G/C=0.70/(0.54+0.70)=0.51$. The sum of the G/C's checks the requirement of 0.90 for 2-phase control, allowing for 0.10 for amber periods.

Part 4.—Here the required solution is for the width of approach B when the approach A width and the intersecting volumes are given. The procedure through the chart is continuously from left to right, using a design capacity setting on the y-y axis. The steps are diagramed in the schematic of figure 27(4). Enter chart at upper left with $W_A=43$ feet for a 2-way street with parking and approach volume of 1,200 v.p.h. This is shown by course *a-b-c-d*, intercepting $G/C=0.45$ at *d*. Locate point *u* on a reading of 1.00 on the V/C_D scale for $MP=750{,}000$, and project horizontally to the right to intersect point *v* on the y-y axis. Draw line *dw* through point *v* intercepting a value of 0.54 on the right G/C scale. Proceed horizontally from *w* to *x*, turning on $V=2{,}200$. Continue vertically to 1-way, no parking curve and turn on point *y*. Then proceed to the right and read $W_A=44$ feet at point *z*. Actual signal timing is: for approach A, $G/C=0.45/1.10=0.41$; and for approach B, $G/C=0.54/1.10=0.49$.

Problem 40

In the *CBD* of a metropolitan area having a population of 1 million, a 2-way, 40-foot street intersects a 2-way 56-foot street. There is no parking on either street. The latter uses the center 12-foot lane for opposing left turns, while the remaining 22-foot approach accommodates through-plus-right movements. The critical approach, A, on the 40-foot street carries a traffic volume of 650 v.p.h. The critical approach, B, on the 56-foot street accommodates 800 v.p.h. of which 200 v.p.h. use the exclusive left-turn lane on a separate signal indication. Other conditions are assumed to be average. Determine the adequacy of the intersection and signal timing.

Solution: Although a heavy left-turning movement is to be accommodated, chart 19 can be used by assuming an additional average G/C of 0.20 for the third signal phase. For approach A enter chart on left with $W_A=20$ feet and, using 2-way, no parking curve and $V=650$, intercept an initial G/C of 0.47 as shown by course *a-b-c-d* in figure 28(1). For approach B enter chart on right with $W_A=22$ and, using 2-way, no parking curve and $V=800$, intercept an initial G/C of 0.51, course *e-f-g-h*. Adjusted initial G/C for approach B is $0.51+0.20=0.71$, as indicated by a vertical shift from *h* to *i*. Connect the two G/C's, 0.47 and 0.71, by a straight line *di*. From the intersection point, *j*, on the y-y axis, project a line horizontally to the left to intercept the V/C_D scale for MP of 1 million population; at point *k* read $V/C_D=1.12$. In table 3, find *f* values for approaches A and B combined, as follows: level C, 1.00; level D, 1.14; level E (possible capacity), 1.20. The intersection, therefore, can accommodate the indicated volumes at level of service D. The required G/C's for operation at level of service D are determined by adjusting the initial G/C values for metropolitan size and for V/C_D ratio; that is, the initial G/C for each phase, in this case, is divided by 1.15 and by 1.12. For approach A, $G/C=0.47/(1.15\times1.12)=0.36$; for approach B, through-plus-right movement, $G/C=0.51/(1.15\times1.12)=0.39$; and for approach B, left-turning movement, $G/C=0.20/(1.15\times1.12)=0.15$. Allowing 10 percent of the cycle for amber, the sum $(0.36+0.39+0.15+0.10)$ totals 1.00.

Problem 41

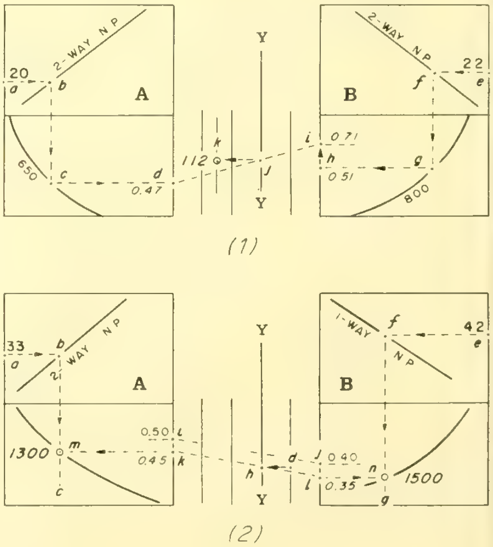
A 66-foot, 2-way street intersects a 42-foot 1-way street in the residential section of a metropolitan area of 100,000 population. There is no parking on either street. Conditions are assumed to be average. If 50 percent of the cycle is to be devoted to green on the 66-foot street and 40 percent of the cycle on the 42-foot street, what volumes can be accommodated on each facility at design capacity and at possible capacity?

Solution: For approach A, enter chart 20 with $W_A=33$ and turning on the 2-way, no parking curve, project downward through the approach volume curves, as indicated by course *a-b-c* in the schematic of figure 28(2). For approach B, enter chart with $W_A=42$ and turning on the 1-way, no parking curve, project downward through the approach volume curves, as shown by course *e-f-g*. Locate point *d* at a reading of 1.00 on the V/C_D scale for MP of 100,000 population. Project to the left horizontally intercepting the y-y axis at point *h*. Through *h* draw line *kl* parallel to line *ij* (where line *ij* connects $G/C=0.50$ on the left scale and $G/C=0.40$ on the right scale, the signal split as per problem statement). From point *k* project horizontally to the intersection point *m* on line *bc* previously established; read $V=1{,}300$ v.p.h. on approach A. From point *l* project horizontally to the intersection point *n* on line *fg* previously established; read $V=1{,}500$ v.p.h. on approach B. Dividing the intercepted G/C values of 0.45 at point *k* and 0.35 at point *l* by the MP adjustment factor of 0.90, yields the required G/C values of 0.50 and 0.40 on approaches A and B. The approach volumes which the intersection can handle at possible capacity are the volumes found above corresponding to design capacity multiplied by the *f* factor in table C of chart 20. Possible capacity: approach A, $1{,}300\times1.22=1{,}600$ v.p.h.; approach B, $1{,}500\times1.13=1{,}700$ v.p.h.

Problem 42

In the *CBD* of a metropolitan area of ½ million population, Main Avenue, a 2-way arterial, is to be improved from a 62-foot street with parking to a facility with two 34-foot traveled ways, without parking, and a 14-foot median including separate left-turn lanes. The existing conditions as well as the proposed improvements of Main Avenue, together with a series of five intersections, are shown in the upper part of figure 29. The peak-hour traffic projected for the improvement is indicated at the critical approaches on the plan. As part of the planning process and preliminary design of a street improvement program, determine first the adequacy of the intersections with Main Avenue improved and cross streets unaltered. Then, as a second step, determine the improvements that are also required on the cross streets to provide operation at level of service C. Type of operation and parking condition on each cross street is to be retained. Generally, widening is to be kept to a minimum; however, approximate limits to which the streets may be widened curb-to-curb, if required, are: 20th and 21st Streets, 44 feet; 22d and 24th Streets, 50 feet; and 23d Street, 68 feet. To save space, the latter may be an odd number of lanes with the center lane at the intersection reserved for left turns.

Solution: The analysis, using chart 19, for the condition where Main Avenue is improved to two 34-foot traveled ways and a 14-foot median with left-turn lanes, while the cross streets remain unaltered, is shown in the left part of the tabulation in figure 29. Future traffic, representative of the p.m. peak, is applied to the plan. Although average conditions are assumed in this type of analysis, adjustment in approach volumes should be made where turns take place on separate lanes. In such instances, the deduction should be of the order of 10 percent of the approach volume or 100 v.p.h., whichever is smaller. A deduction of 100 v.p.h. has been applied generally, as shown in the tabulation, on Main Avenue approaches at 21st, 22d and 24th Streets. At 23d Street, the left-turning movement in the northwest quadrant is sufficiently heavy to require a separate signal indication. The left-



(1)

(2)

Figure 28.—Chart solutions for problems 40 and 41.

TABLE C — RATIO OF POSSIBLE CAPACITY TO DESIGN CAPACITY

STREET TYPE	PARKING CONDITIONS	FACTOR f, when W_A (in feet) is —								
		10	15	20	25	30	35	40	50	60
TWO WAY	No Parking	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
	With Parking	–	–	1.10	1.14	1.18	1.21	1.25	1.31	1.34
ONE WAY	No Parking	–	–	1.15	1.13	1.12	1.12	1.13	1.15	1.17
	Parking One Side	–	–	1.10	1.13	1.16	1.18	1.20	1.25	1.30
	Parking Both Sides	–	–	–	1.25	1.25	1.25	1.27	1.32	1.37

EXAMPLE

Given

MP = 750,000, O.B.D.

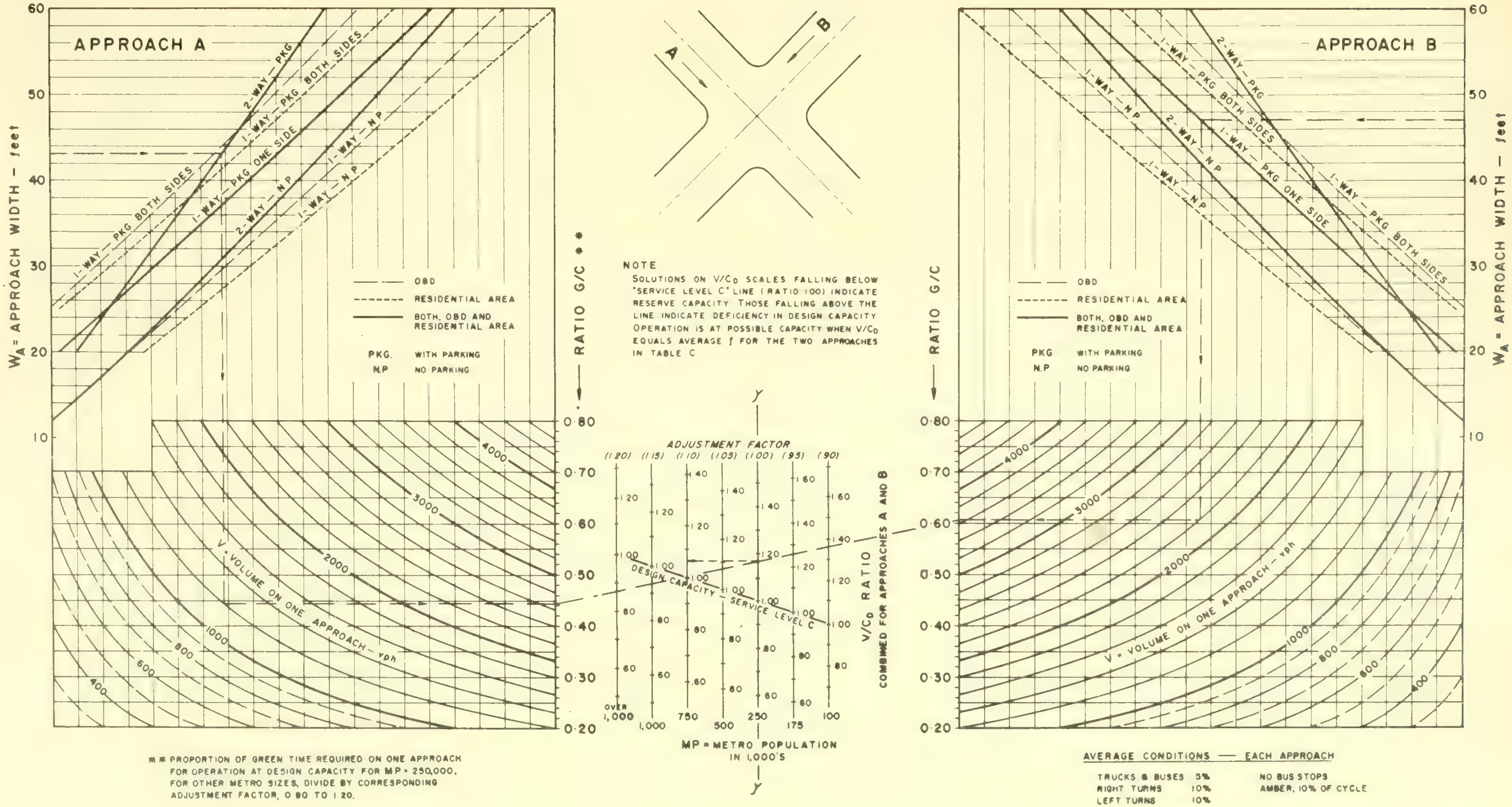
APPROACH A:
Two-way
With parking
W_A = 43'
DHV = 1200 vph

APPROACH B:
One-way
Parking one side
W_A = 47'
DHV = 2200 vph

Solution

V/C_D = 1.06

Operation inferior to service level C; i.e., demand volume exceeds design capacity, but well within possible capacity for which the ratio would be 1.25 (average f for the two approaches in Table C).



NOTE
SOLUTIONS ON V/C_D SCALES FALLING BELOW "SERVICE LEVEL C" LINE (RATIO 1.00) INDICATE RESERVE CAPACITY. THOSE FALLING ABOVE THE LINE INDICATE DEFICIENCY IN DESIGN CAPACITY. OPERATION IS AT POSSIBLE CAPACITY WHEN V/C_D EQUALS AVERAGE f FOR THE TWO APPROACHES IN TABLE C.

** PROPORTION OF GREEN TIME REQUIRED ON ONE APPROACH FOR OPERATION AT DESIGN CAPACITY FOR MP = 250,000. FOR OTHER METRO SIZES, DIVIDE BY CORRESPONDING ADJUSTMENT FACTOR, 0.90 TO 1.20.

AVERAGE CONDITIONS — EACH APPROACH

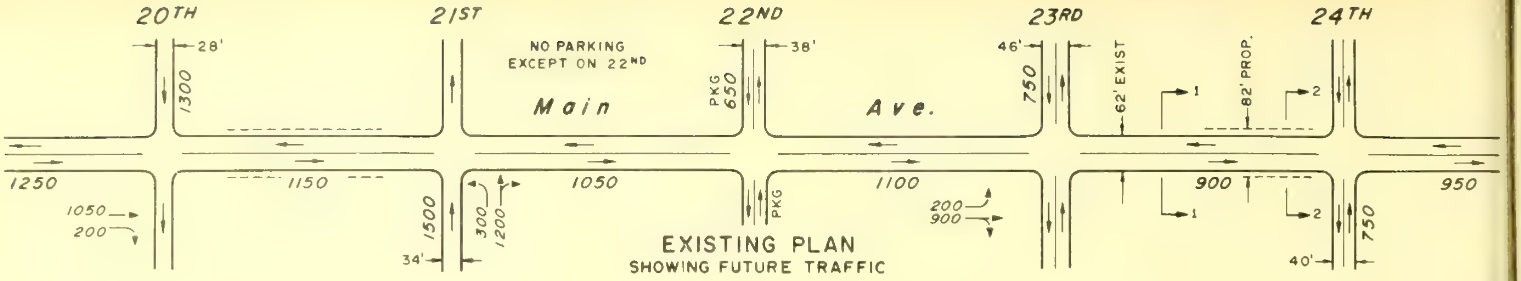
TRUCKS & BUSES	5%	NO BUS STOPS
RIGHT TURNS	10%	AMBER, 10% OF CYCLE
LEFT TURNS	10%	

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

OVER-ALL INTERSECTION CAPACITY — AVERAGE URBAN CONDITIONS
O.B.D. AND RESIDENTIAL AREA

CHART 20

20
O.B.D.
RES.



CAPACITY ANALYSIS FOR WIDENED MAIN AVENUE

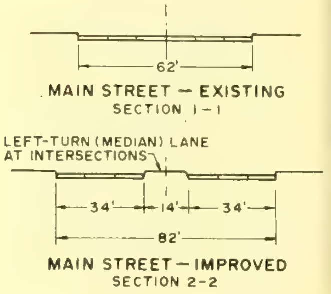
CROSS STREETS — EXISTING							CROSS STREETS — IMPROVED	
CROSS STREET		W_A feet	DHV — vph ONE DIRECTION		V/C_D **	f	W_A — APPROACH WIDTH — feet FOR LEVEL OF SERVICE C	
LOCATION	TYPE		MAIN *	CROSS ST.			REQUIRED	SELECTED
20TH	1-WAY, NP	28	1250	1300	1·17	1·17	*39*	44 (44)
21ST	1-WAY, NP	34	1050	1500	1·07	1·17	*40*	44 (44)
22ND	2-WAY, PKG	19	950	650	1·14	1·16	*25*	25 (50)
23RD	2-WAY, NP	23	900 ɸ	750 †	1·12	1·21	*25*	24 (60) ♀
24TH	2-WAY, NP	20	800	750	0·93	1·21	*18*	20 (40)

* LEFT-TURN MOVEMENT ON SEPARATE LANE REMOVED: 200 vph AT 23RD ST; 100 vph AT 21ST, 22ND AND 24TH STS.
ɸ APPROACH REQUIRES THIRD PHASE (ASSUMED G/C = 0·20) FOR LEFT TURN.
† 650 ON IMPROVED CROSS STREET, ALLOWING FOR 100 vph ON SEPARATE LEFT-TURN LANE.
♀ 24+12+24; CENTER LANE FOR LEFT TURNS
** ANALYSIS BASED ON W_A = 34' ON MAIN AVE. WITH LEFT-TURN LANES.

METRO POPULATION — 500,000
LOCATION — CBD

TRAFFIC SHOWN:
DHV — PM PEAK

AVERAGE CONDITIONS ASSUMED EXCEPT FOR TURNING MOVEMENTS AT: 20TH-21ST ST. COUPLE, SW QUAD; AND AT 23RD ST, NW QUAD



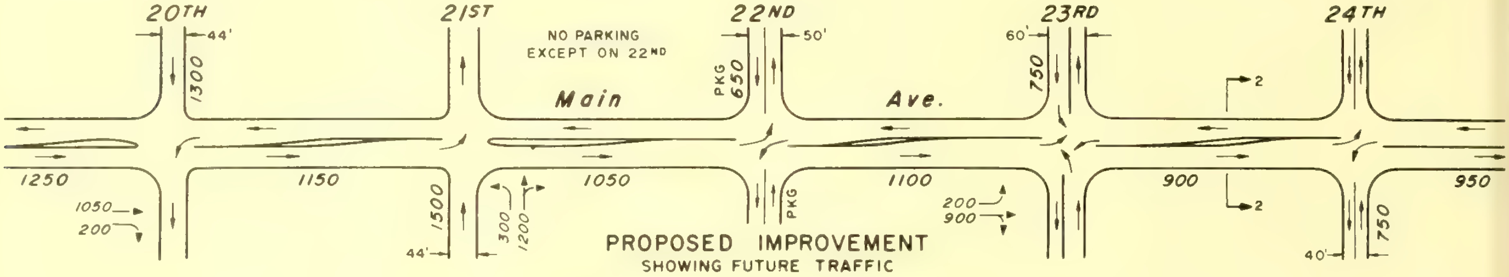


Figure 29.—Problem 42 illustrated.

turning volume of 200 v.p.h. is deducted from the approach volume and an additional G/C of 0.20 is assumed in the solution of chart 19.

With the information listed in the first five columns of the tabulation in figure 29, the V/C_D values were found readily in chart 19 and recorded in the sixth column. In the seventh column, the possible capacity ratios, or f factors, were obtained for the same intersections from table C in chart 19. All V/C_D ratios exceed 1.00, indicating that design capacity would be surpassed, except at 24th Street. Comparison of V/C_D values with the f factors clearly indicates the degree of overloading beyond level of service C. Intersections at 20th and 22d Streets would operate at approximately the possible capacity, and intersections at 21st and 23d Streets would operate at an intermediate level between design and possible capacities. The cross streets at these intersections require improvement along with Main Avenue to accommodate the future traffic. The intersection at 24th Street, on the other hand, shows a V/C_D ratio of less than 1.00, indicating that no widening on the cross street is required.

The proposed cross-street improvements are shown in the last two columns of the tabulation and in the lower plan of figure 29. The required widths of cross streets were taken directly from chart 19, using the indicated volumes, a 34-foot approach on Main Avenue, and a design capacity control. The selected widths are rounded values predicated on lane widths of 11 feet or more, within the indicated limits of permissible maximum widening. Twenty-fourth Street was left unaltered because of available capacity reserve.

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Harry C. Secrest, *Managing Editor*

Fran Faulkner, *Editor*

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ERRATA

n the first installment of this article, published in the August 1967 ssue of PUBLIC ROADS, *A Journal of Highway Research*, vol. 34, o. 9, an error has been noted in charts 3 through 15. In the last line f the note at the top of each of these charts, please change *item 5* o read *page 198*. Also, in chart 10, under example, Pkg *left* side hould read Pkg *right* side.

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DECEMBER 1967

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Snow removal operations on Fremont Pass, Colorado

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Snowdrift Control Through Highway Design

Y THE
UREAU OF PUBLIC ROADS

Reported by **FREDERICK W. CRON,**
Regional Design Engineer,
Region 9, Denver, Colorado

In areas that are subject to large and frequent snowfalls, drifting snow on highways presents a serious maintenance problem. In the article presented here, the author discusses the research that has been accomplished in this field and presents recommendations for dealing with the problem of snowdrift control. In many areas of the country, particularly the northern States, the maintenance costs of highways during the snow season can be reduced considerably if the problem of drifting snow is considered when a highway is in the design stages. Research shows that where topographic conditions are favorable the sweep of the wind can be used to control the accumulation of snowdrift on highways. In areas where blowing snow is a problem, information on prevailing winds and drifting should be considered in the selection of a highway location. The profile and cross sectional design can also provide snow control by taking advantage of the wind and should be considered when a highway is initially planned or when reconstruction for snow control is planned. Appurtenances such as curbs, guardrails, fences, signs, etc., must also enter consideration for snow control because they are obstacles to the free movement of the wind and therefore contribute to the formation of drifts. By generally streamlining the area surrounding the highway, drifting is minimized, the esthetic qualities are enhanced, and the operational safety is increased.

Introduction

N THE NORTHERN STATES the drifting of snow presents a serious highway aintenance problem and expenditures for lowdrift control and snow removal run into le millions every winter. In some places xpenditures are so large that attention lould be given to snowdrift elimination in le design stage. The increased construction ost of roads designed for snow control is to large degree offset by decreased maintenance osts.

In Europe, North America, and Japan iere is a considerable body of literature on le control of drifting snow, most of which as appeared since 1950. Mellor's bibliograhy of November 1965 (*1*)[1] is the most recent ompendium of the available literature. chneider (*2*) also cites an extensive list of ources extending back to work by Schubert 1903. E. A. Finney pioneered research in le field at Michigan State College in the iddle and late thirties, using balsa sawdust id flake mica to simulate snow. He studied le performance of various highway cross ections and snow fence designs in a wind unnel. The practical validity of his findings as never been seriously challenged (*3*, *4*).

onclusions and Recommendations

With the information now available new ighways can be designed that will, to a rge degree, use the sweep of the wind to revent drifted deposits of snow on the roaday. The maintenance expense thus saved ill largely offset the increased construction ost of the snow-control design. Some existg highways can be reconstructed at moderate ost to reduce drifting, primarily by flatteng and rounding slopes and removing driftaking obstacles on and near the right-of-way. or many miles of existing highway, however, le main defense against drifting from blowg snow will continue to be systems of snow ences, hedges, or windbreaks.

The author believes that additional reearch is needed to determine ways to protect iterchanges and their associated ramps and ructures from drifting snow. Control of the ground blizzards that occur in the Great Plains area should also be investigated, particularly the relationship between these blizzards and grazing. Reduction or elimination of grazing in a zone 500 feet wide upwind from the highway might help to control ground blizzards. Such zones could be established as scenic easements to the benefit of both esthetics and snow control. There is also a need for information on the cost of drift control and removal as related to highway design, particularly before and after cost figures such as those cited in this article for South Dakota. Such figures are needed to support the increased construction expenditures required to achieve built-in drift control by use of higher gradelines, flatter slopes, and streamlined cross sections.

Highway Location

In regions where blowing snow is a serious problem, snow control should be a prime consideration in locating the highway. Reconnaissance should be made during the winter months, as well as at other times of year, and in suspect areas all available information on prevailing winds and drifting should be collected and analyzed before the highway location is decided (*5*).

Where a choice of location is possible, considerable immunity from snowdrift formation can be achieved by proper selection of the road location. Valley locations through timber have few drift problems because the trees shelter the road from the wind and trap the snow (*4*). In rolling country the leeward side of natural slopes should be avoided, and the road should be constructed on the windward side or on the crest if possible (*4*, *5*). For minor roads in heavy snow country considerable indirection of travel can be justified to stay on the ridges, as this may mean the difference between keeping the road open or having it closed for days awaiting the county snow plows. Many county roads in Wyoming deviate considerably from the usual section-line location to follow ridges and thus avoid severe drifting.

Drift Control by Profile Design

In open level country the wind can be used to sweep the highway clear of snow if the gradeline of the highway is raised above the adjacent ground level. The amount of elevation can be determined from snowfall records. To be effective, the finished grade should be higher than the depths of the snow deposits adjacent to the highway and higher than the prevailing adjacent vegetation (*4*). Michigan, Minnesota, South Dakota, and other States that have difficult snow problems try to elevate gradelines 3 to 5 feet above the adjacent ground.

Significant decreased winter maintenance expenditures have resulted from proper design. In South Dakota, Route U.S. 83

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[1] Italic numbers in parentheses indicate the references sted on page 234.

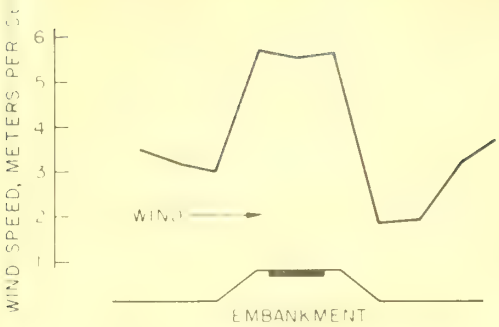

Figure 1.—Distribution of wind speed across an embankment (Benterud, 1947).

between Vivian and Fort Pierre was reconstructed in 1960 to provide an elevated profile and streamlined cross section. Before reconstruction it had a low profile, sharp slopes, and small ditches, and the snow removal costs averaged from $488 per mile in a severe winter to $203 per mile in a moderate winter (*6*). In the 6-year period since reconstruction the average annual snow removal cost for this highway has been only $68 per mile.

Such savings can repay in a few years the cost of the additional earthwork required for snow control. On one 4-lane Interstate highway project in South Dakota, snow control was achieved at an increased construction cost of only $6,000 per mile (*6*).

In Colorado an elevated gradeline did not entirely eliminate drifting on wide-median divided highways when both roadways were constructed at the same level. However, by elevating the downwind roadway slightly above the upwind roadway the self-clearing properties of the highway cross section were restored. This design was used for Interstate Highway 80S from Brush to Sterling, Colo., completed in 1965.

Cross Sectional Design

An elevated roadway or embankment is a barrier to the free sweep of the wind and causes eddies to form on both the windward and leeward sides of the road. The wind velocity in these eddies is less than that of the main airstream, permitting deposition of some of the suspended burden of snow to form drifts as shown in figure 1. Since the wind's power to transport snow varies as the cube of the velocity, even small decreases in velocity will cause snow to be deposited in drifts. The size, shape, and location of the drifts are influenced by the cross sectional shape of the roadway. Finney determined that the major eddy occurs on the leeward side of an obstacle, that the length of the eddy is about 6:5 times the height of embankment, and that wind velocity has no effect on the shape or length of the eddy. Further, the boundary of the eddy area is constant for all embankment slopes up to about 6½:1 at which ratio the eddy almost disappears as shown in figure 2. Finney also learned that when the windward slope of a highway embankment is steep the airstream is deflected upward at the shoulder line and creates a slight eddy over the roadway which could cause drifting. This eddy disappears when the side slopes of the highway embankment are 4:1 or flatter as shown in figure 3.

The findings of Finney were supported by similar findings obtained from full-scale tests in Michigan by the U.S. Army Snow, Ice, and Permafrost Research Establishment in 1955 and 1956. Test embankments were 6 feet high with 2:1, 3:1, and 4:1 side slopes. Drifts accumulated on all embankments until th effective side slope was reduced to 9:1, shown in figure 4, after which no furth drifting occurred. At this time the level dept of snow in the adjacent fields was 21 inche A 3-foot embankment at the test site was n completely self-clearing when 27 inches snow lay on the adjacent fields, illustrating th importance of an adequate difference i elevation between the embankment grade an the general level of the snow cover on eac side (*1*).

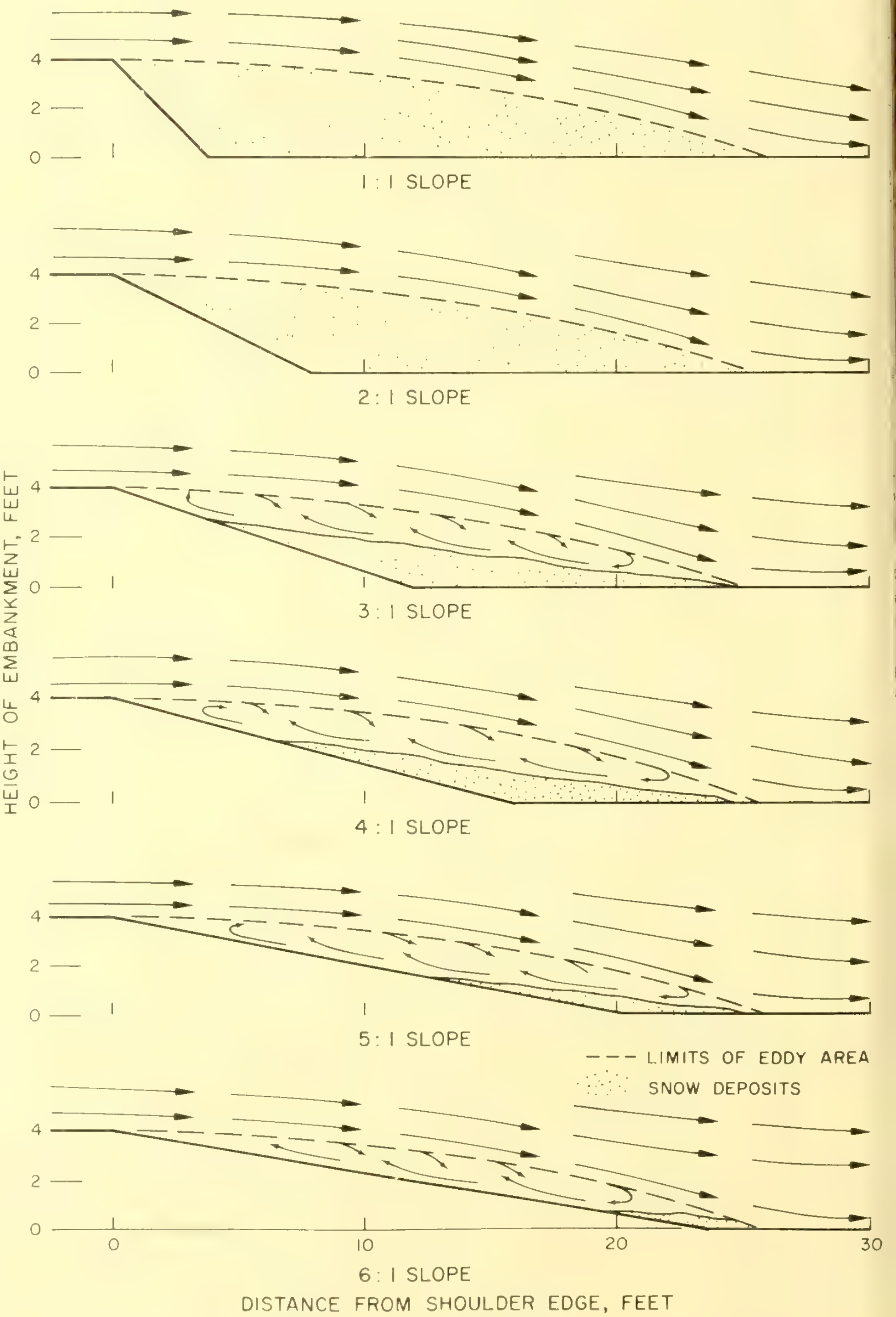

Figure 2.—Effect of embankment slope on drifting (Finney, 1939).

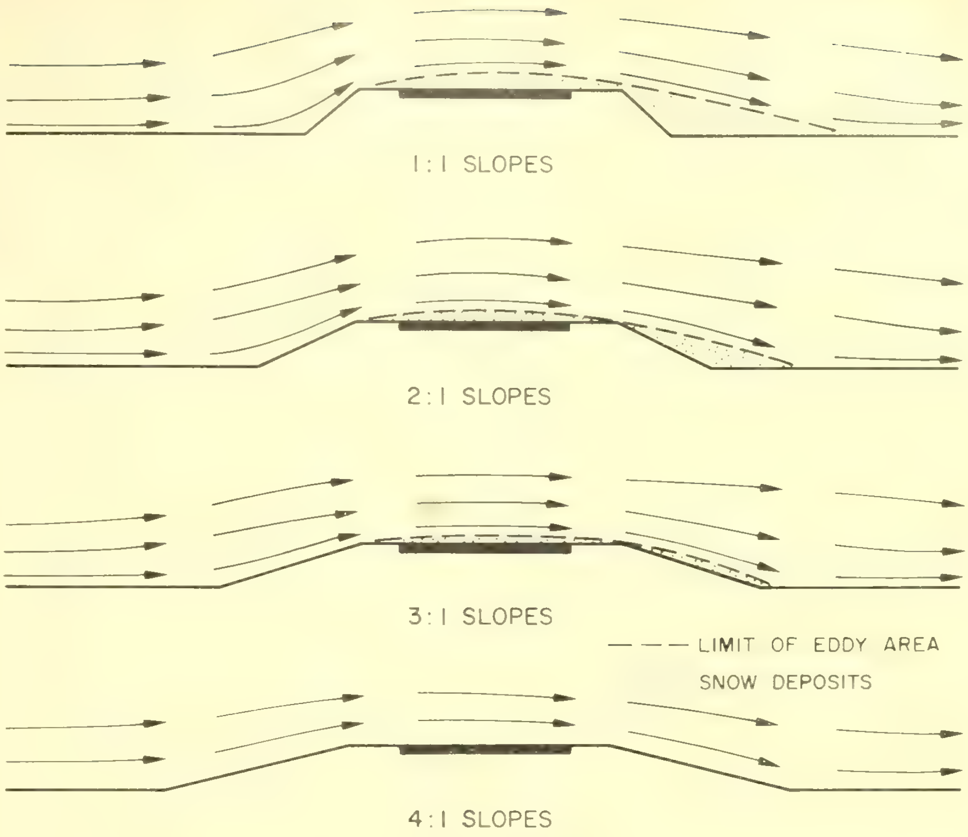

Figure 3.—Airflow across embankments with different slopes (Finney, 1939).

Rounding the intersection of the slope line t the shoulder and toe of the fill does not hange the size or shape of the eddy area, but oes cause the drift to form nearer to the toe f the slope as shown in figures 5 and 6. Vhen the fill is rounded the eddy area starts t the point where the vertical curve is tangent o the shoulder plane, but in a cut the eddy tarts at the point where the vertical curve is ngent to the original ground slope (*4*).

The conditions observed on the leeward ide of a highway embankment also exist where he highway is in cut, except that the eddy rea is above the highway grade and upwind om it, as shown in figure 7. The length of he eddy area is approximately 6:5 times the epth of cut. Where the cut slope approxiates the boundary of the eddy area, the wind tream sweeps the slope area clear and snowrifts do not form. If, however, the slope is teeper than the boundary of the eddy, the ddy area is eventually filled solid with nowdrifts, and no further drifting occurs. hus, for the pavement and shoulders to emain free of snowdrifts, the horizontal istance from the shoulder edge to the top f cut must be at least 6:5 times the depth f cut, as shown in figure 8, (*4*). Where the now supply is ample, unwidened cuts can be uickly filled by drifts, the characteristic lope of the final surface being 8:1 on the pwind side of centerline and 6:1 downwind, s shown in figure 9, (*2*). Cuts more than bout 25 feet deep are less vulnerable to rifting than shallower cuts because there is ore room on the side slopes to hold the indborne snow deposits (*2*), as shown in gure 10.

There is general agreement that shallow uts should be graded with backslopes no teeper than 6:1 and flatter if possible. idehill cuts should be daylighted on the wer side. Depressed medians should have lopes no steeper than 10:1 to facilitate the weeping action of the wind.

Deep cuts should have wide ditches to rovide storage not only for windborne snow ut also for snow plowed from the roadway. Vhere snow-bearing winds are predominantly rom the same direction, most of the extra itch width should be placed on the upwind ide of the road. Finney suggests that all cuts e routinely widened and the resulting aterial be used to raise embankments and atten fill slopes (*4*). Some of the snow tates, notably South Dakota, obtain borrow y widening cuts on the upwind side, thus providing snow storage space and embankment material in one operation. If the grading is skillfully handled and ample rounding provided, such borrowing can be accomplished with a satisfactory overall appearance. Usually, however, extra right-of-way width is required.

The Effect of Highway Appurtenances on Drifting

Highway appurtenances, such as curbs, guardrails, right-of-way fences, traffic signs, delineators, lighting standards, and signal supports contribute to the formation of drifts because they are obstacles to the free movement of the wind. Eddies form behind each obstacle and some of the snow carried by the wind is dropped into the eddy areas as shown in figure 11.

Delineators and guideposts, to the extent that they interfere with plowing and result in windrows of snow on the shoulder of the roadway, contribute to drifting on the pavement and shoulders. Curbs also cause drifting on the pavement, particularly shoulder curbs on fills, because they interfere with the smooth flow of the wind over the fill contour and cause eddies to form.

Guardrails, especially those of the beam type, contribute to the formation of long continuous drifts along the roadway. At the same time the guardrail interferes with snow plowing and causes a continuous windrow of snow to accumulate against the rail posts, closing the opening under the beam. The guardrail then performs as a 3-foot solid snow fence creating an eddy zone downwind for a distance of about 45 feet, which may cause drifting over the entire width of a 2-lane road as shown in figure 12.

The best way to avoid drifting caused by guardrails is to eliminate the guardrails, and where fill slopes are as flat as 6:1 this can be

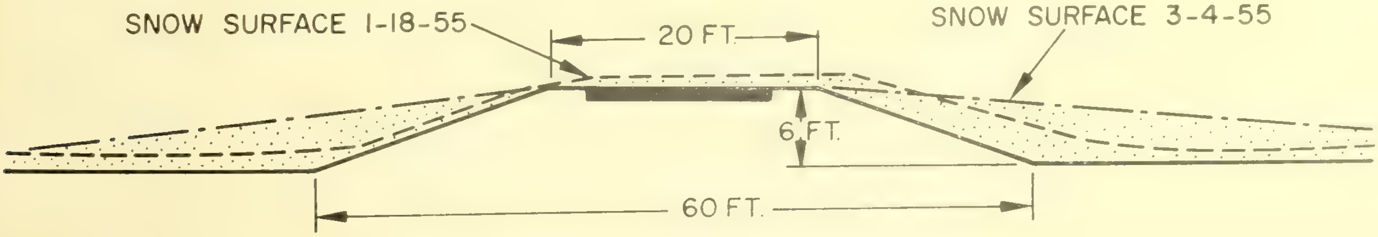

Figure 4.—Drift formation against a test embankment (Gerdel, 1960).

done without sacrificing operational safety. The current safety practice of moving sign supports, lighting standards, utility poles, and other fixed obstacles out from the roadway also contributes to reduction in drift formation.

Anything that contributes to the roughness of the surfaces exposed to the wind can cause snow accumulation. In South Dakota the surface roughness resulting from using a $\frac{3}{4}$-inch chip for surface treatment caused snow to deposit in the irregularities, which was then packed by traffic making the road extremely slippery. (*6*).

The most difficult problems with windblown snow occur at interchanges and grade separations. In snow country it is usually best to carry the more important highway over the crossroad, so the wind can sweep it clear of snow. However, much of the windblown snow will then collect around the bridge structure and in the depressed crossroad and ramps. Some relief from the snow accumulation may be provided by streamlining the interchange grading to avoid steep slopes and sharp angles; however, in many instances the most effective treatment will be to intercept the snow by snow fences, windbreaks, or hedges before it reaches the interchange.

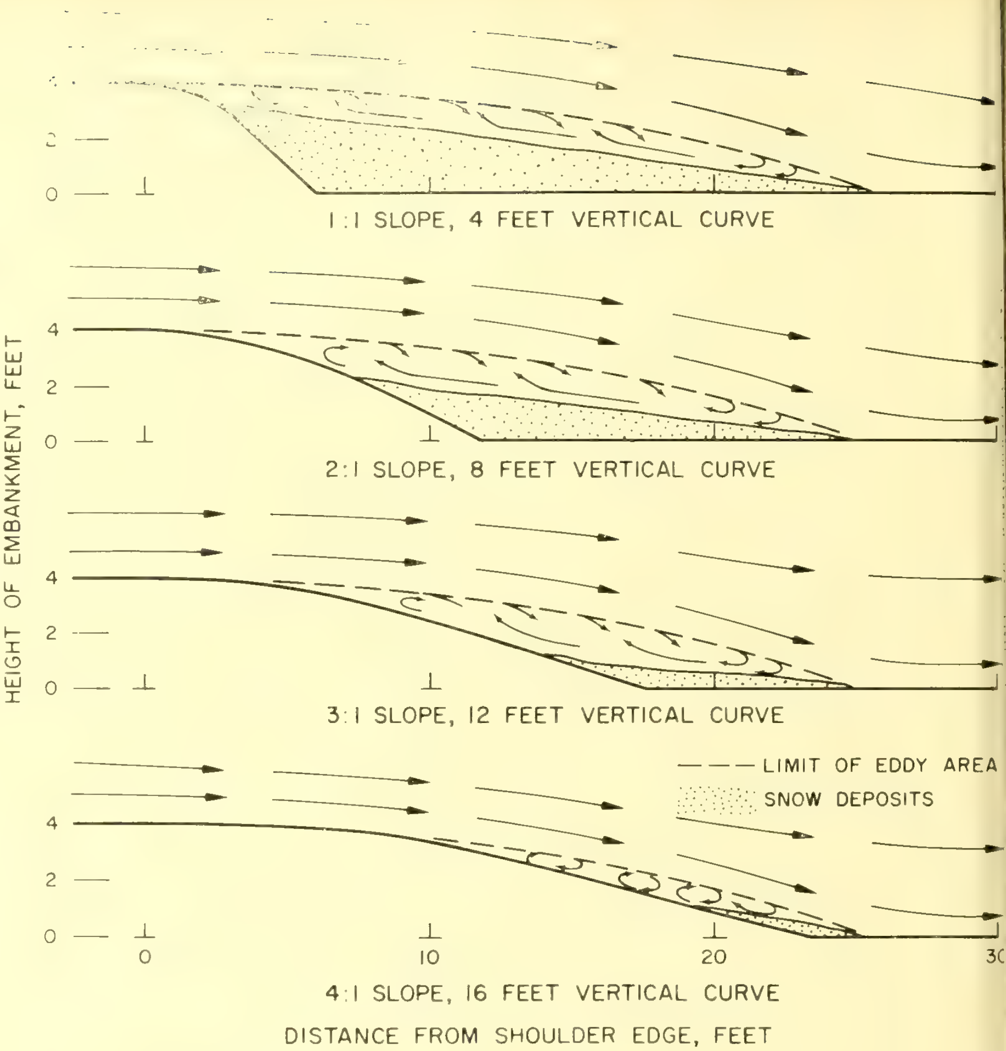
HEIGHT OF EMBANKMENT, FEET
2
0
1:1 SLOPE, 4 FEET VERTICAL CURVE
4
2
0
2:1 SLOPE, 8 FEET VERTICAL CURVE
4
2
0
3:1 SLOPE, 12 FEET VERTICAL CURVE
LIMIT OF EDDY AREA
SNOW DEPOSITS
4
2
0
0
10
20
3
4:1 SLOPE, 16 FEET VERTICAL CURVE
DISTANCE FROM SHOULDER EDGE, FEET

Figure 5.—Effect on drifting of rounding top of slope (Finney, 1939).

Roadside Development and Landscaping

A generation ago the Bureau of Public Roads urged the adoption of a streamlined cross section for highways with comparatively flat slopes, rounded and warped to merge into the natural contours of the adjoining terrain, and with broad rounded ditches in place of the V-ditches used at that time (*7*). This cross section was promoted principally because of its esthetic advantages, but it also had improved properties for snow control. Grading for landscaping and grading for snow control complement each other because both result in desirable streamlining of the roadway cross section. Flat cut and fill slopes, desirable for snow control, are easier to stabilize by grass cover and easier to mow after the cover is established. Mowing in turn contributes to easier snow control in the winter. The streamlining of interchange grading improves appearance and reduces the eddy-forming angularities that cause snow to deposit. Both purposes are served by preparing contoured grading plans to control the field grading.

When it comes to plantings on or adjacent to the highway, landscaping should be closely coordinated with snow control to avoid the creation of maintenance problems. At one extreme are those sections of highways that lie in sheltered valley locations or through woodlands. Drifting snow is seldom a problem in these areas so cuts may be planted with shrubs and trees, glare shields may be planted in the medians, and accent planting may be used at structures and interchanges. At the other extreme are highways in the windswept grasslands of the high western plains where there is little or no vegetation other than grass. In most of these areas the planting of trees and shrubs on or immediately adjacent to the highway will cause or contribute to drifting and should be avoided. In particular, plantings should not be made in medians or on the slopes upwind from the roadway. Fortunately, in treeless areas restraint in planting is also good esthetics for such plantings are incongruous in the prairie setting. Between the extremes an infinite number of variations exist, and the decision to plant and what to plant should be made only after study of all aspects of operation and maintenance, including snow control and operational safety. The removal of trees from the immediate vicinity of the traveled way for safety reasons also reduces the possibility of drifts forming leeward from the trunks, and so contributes to snow control.

In some areas, notably Minnesota, windbreaks of trees or living snow fences have been planted to control drifting on the highways. Some of these windbreaks are so effective that they intercept practically all snow except that falling directly on the highway. Other States are also planning to make extensive use of shelterbelts located a safe distance upwind to control drifting snow.

Snow Control and Operational Safety

The present trend in highway design is to clean up the roadsides by removing fixed obstacles that could cause damage if struck by vehicles out of control. Fill slopes are flattened wherever possible so the driver of a vehicle leaving the road may have an opportunity to recover control without capsizing. The flattened fill slopes provide a streamlined cross section and obviate the need for guardrails. It is becoming common practice to use 6:1 slopes for heights of fill up to 16 feet; in bad snow country 6:1 slopes could be justified for even higher fills.

The use of shoulder curbs for erosion control is a questionable practice for both snow control and operational safety. Not only do shoulder curbs cause eddies and drifting of snow on the pavement, but they can easily cause a vehicle to flip over if the vehicle strikes the curb at an oblique angle at high speed. Further, in semiarid regions shoulder curbs inhibit growth of vegetation on the slopes by intercepting water from the pavement and shoulders that would otherwise nourish plantlife.

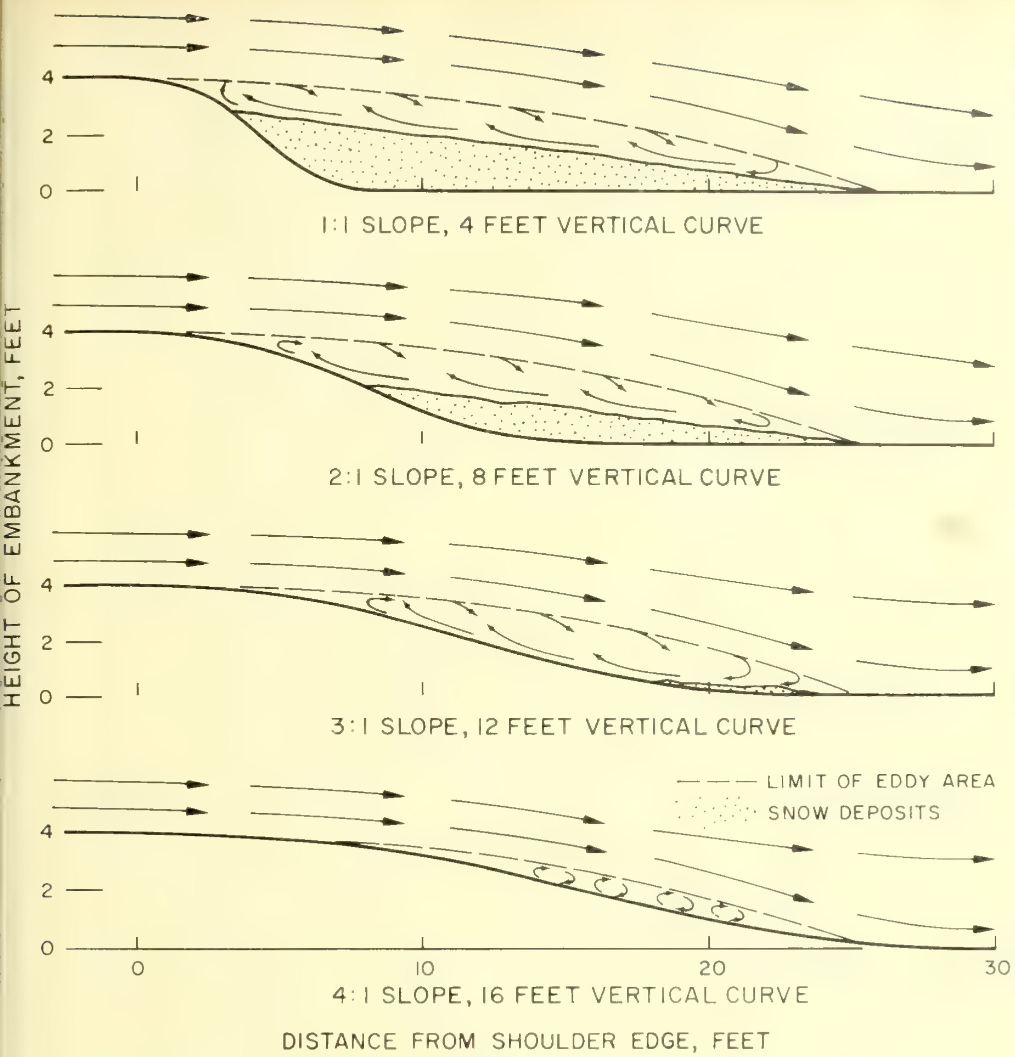
HEIGHT OF EMBANKMENT, FEET
4
2
0
1:1 SLOPE, 4 FEET VERTICAL CURVE
2:1 SLOPE, 8 FEET VERTICAL CURVE
3:1 SLOPE, 12 FEET VERTICAL CURVE
LIMIT OF EDDY AREA
SNOW DEPOSITS
0
10
20
30
4:1 SLOPE, 16 FEET VERTICAL CURVE
DISTANCE FROM SHOULDER EDGE, FEET

Figure 6.—Effect on drifting of rounding top and bottom of slope (Finney, 1939).

The removal of hazards such as large trees, sign supports, and light poles from the immediate vicinity of the traveled way is desirable for reducing snowdrifts and may further contribute to safety by reducing shade on the pavement that might cause icing.

Maintenance Practices

The most effective measures for snowdrift control use the wind to make the roadway self-clearing. However, because the wind does not always blow during a snowstorm, or from the same direction, or with uniform effectiveness, some deposits of snow must be plowed from the road. One pass of the snow plow can destroy the effectiveness of a streamlined elevated gradeline if the plow leaves windrows of snow on the shoulder of the road, especially the upwind shoulder; the windrows cause eddies and drifts rapidly form. To prevent the formation of these drifts the plowed snow should be scraped level with the top of the fill and wasted on the slopes. Shoulder curbs should be avoided because they interfere with clean snow removal. In cuts the plowed snow should be moved as far as possible from the shoulders. In areas particularly susceptible to drifting it may be advisable to pick the snow up with rotary equipment and blow it onto the slopes.

Much can be done before the first snowfall to prepare the highways for winter maintenance. Grass should be mowed as soon as the growing season is over, especially the grass adjacent to the right-of-way fences. Where blowing snow is a problem, ivy, honeysuckle, creeper, and other vegetation growing on the right-of-way fences should be removed so that wind can blow freely through the fences. In the West deposits of Russian Thistle, or tumbleweed, which collect upwind from the right-of-way fences and cause drifting, should be removed or burned. Turnrows caused by plowing adjacent fields should be smoothly leveled off. Maintenance materials stored on the upwind side of the roadway should be removed, transferred to the downwind side, or streamlined to eliminate eddying in their lee. Sand barrels should be placed on the downwind shoulder to minimize drifting. Piles of dirt from ditch cleaning should be spread in thin layers. All obstructions on the roadway that can interfere with the sweep of the wind and cause drifting should be eliminated or streamlined.

Obstructions Outside the Right-of-Way

Some obstructions that cause drifting on the highways are outside the right-of-way and beyond the control of the highway department. These include buildings, advertising signs, private hedges, windbreaks, and shelterbelts that are too close to the road. A natural

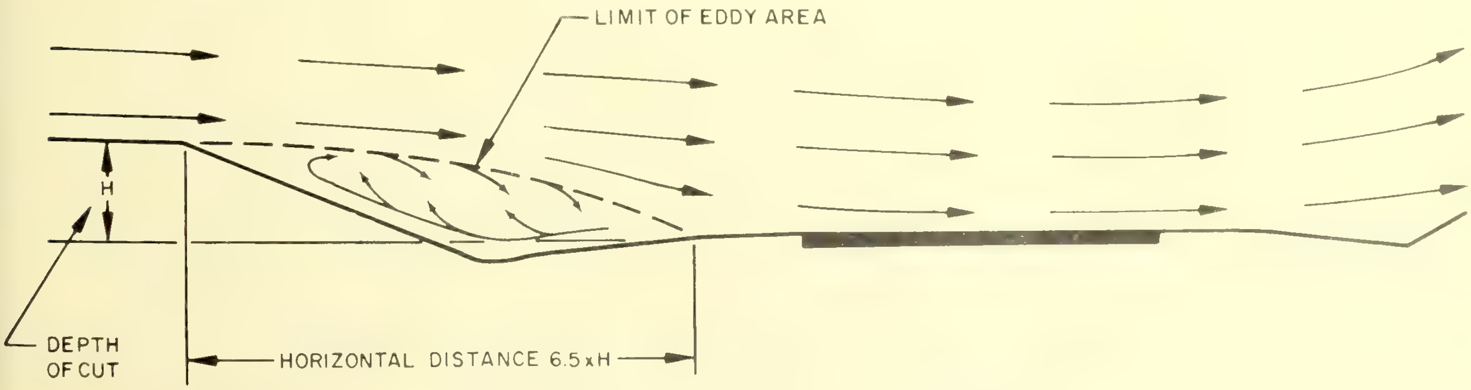

Figure 7.—Drift-forming eddies in a cut (Finney, 1939).

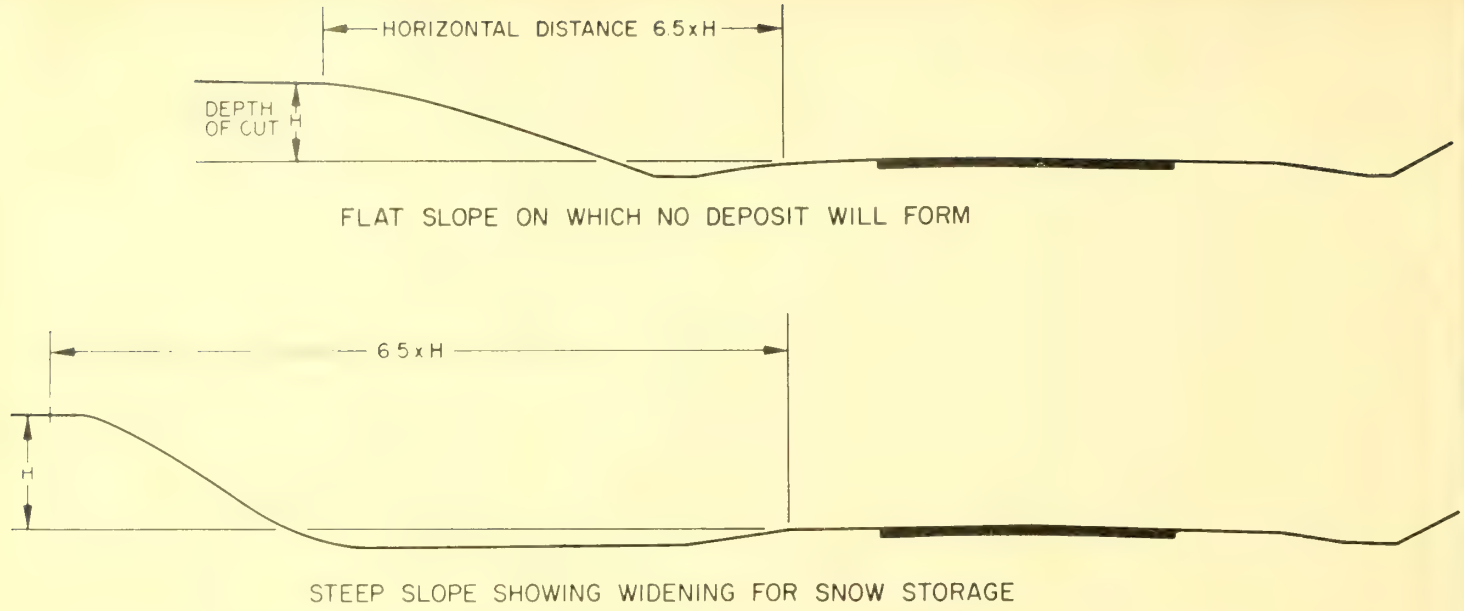
HORIZONTAL DISTANCE 6.5 x H
DEPTH OF CUT
H
FLAT SLOPE ON WHICH NO DEPOSIT WILL FORM
6.5 x H
H
STEEP SLOPE SHOWING WIDENING FOR SNOW STORAGE

Figure 8.—Typical highway cross sections for minimum snow deposits (Finney, 1939).

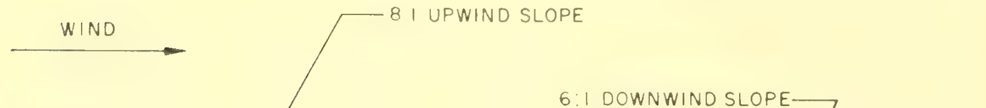

Figure 9.—Successive stages of snow deposits in a deep cut according to Schubert (1903).

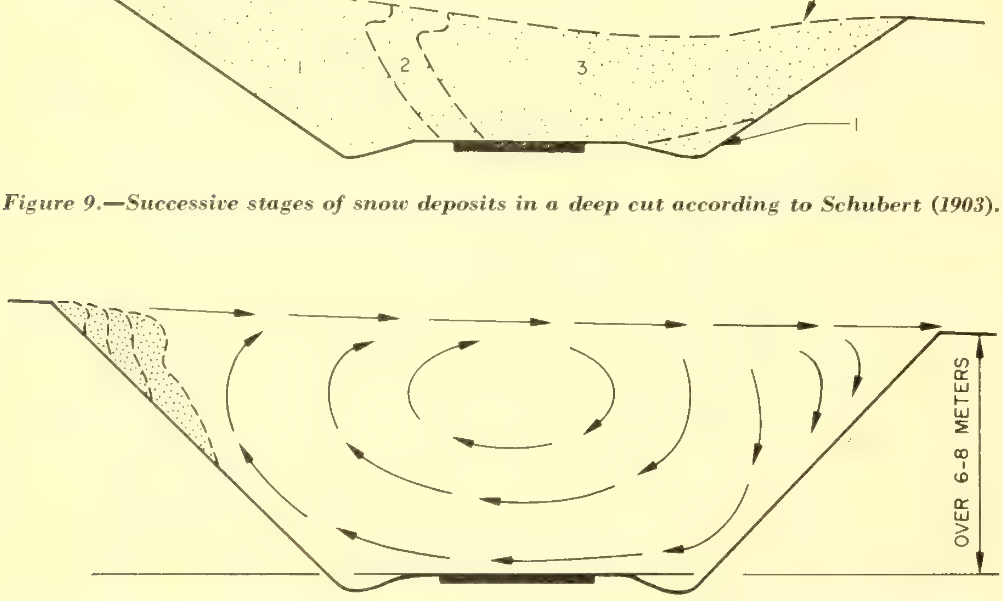

Figure 10.—Successive formation of snow deposits in a deep cut according to Petersen (1942).

topographic ridge upwind from the highway on private land may cause large drifts to form on the road. Generally, little can be done to remove these obstructions from existing highways, but as the roads are reconstructed or upgraded, efforts should be made to correct the worst conditions. Sometimes relocation of the highway is required and usually additional right-of-way width must be acquired. In many counties a systematic program of cleaning and resloping road sides in cooperation with the adjacent landowners will return its cost many fold in reduced winter maintenance (*8*).

Sometimes, the conditions on adjacent private land tend to reduce snow drifting on the highway. For example, the stubble from some crops, especially corn, acts as a reservoir to prevent the movement of snow. A strip of unplowed corn stubble 200 feet wide or wider upwind from the highway w greatly reduce the quantity of blowing sno reaching the highway (*3*). Windbreaks ar shelterbelts, if they are far enough upwin will also prevent large quantities of snow fro drifting on the highway (*9, 10*).

On the High Plains ground blizzards are troublesome phenomenon. Snow carried alor close to the ground by the wind may con pletely obscure the driver's view of the roa even on clear days. The supply of snow fro adjacent closely grazed range lands seen inexhaustible and the blizzard continues long as the wind is strong enough to mo the snow. Delineators and guideposts a necessary to keep drivers from running o the road during ground blizzards, but the guideposts sometimes contribute to drifti on the road as previously described. Groun blizzards are difficult to control, but son measure of relief may be obtained by trappi as much of the snow as possible before reaches the highway with snow fences, sno ridges, or shelterbelts.

Snow Fences

As previously explained, a large measu of drift control can be achieved on new hig ways by selection of the location and b cross sectional and profile design. Howeve the highway locator is seldom able to choos his location for snow control alone, nor can favorable profile and cross section always b attained, owing to other restrictions such a right-of-way. Other measures, such as sno fences, windbreaks, shelterbelts, and eve snowsheds, must therefore be used to contr the drifting snow.

The design and placement of snow fence and windbreaks has been studied intensivel for years, but a full discussion of them wa beyond the scope of the study reported here resent practice seems to favor fences 4 to 6 et high of about 50 percent density, that is,) percent of the vertical area presented to e wind is solid material. The length of the ift formed in the lee of the fence and the antity of snow trapped in front of and hind it are functions of the height of the nce, its density, and the free space between e ground and the bottom of the fence. ences are most efficient when placed at right gles to the wind, as shown in figure 13.

Where space is limited it may be desirable) create snow ridges by successively raising e snow fences before they become buried. Russia ridges or "ramparts" 10 meters high ere produced by using movable wooden anels, called snow shields, to form snow arriers (*2*). In the thirties drifts 27 feet high ere produced by the Bureau of Public Roads this manner in the Colorado Rockies. he proper location of snow fences requires nowledge of the topography along the ighway, of the prevailing winds, and of e past locations of drift-susceptible spots. now fence should be placed at all known rift spots as one or two bad spots left unprotected will decrease the effectiveness of the hole system. The horizontal distance from he roadway to the nearest snow fence depends n the height and type of fence and varies om 10 to 15 feet per foot of fence height. Vhere the supply of snow is large, two or ore parallel rows of fences may be required, ut the rows should be far enough apart that he drifts do not overlap. In irregular terrain may be necessary to experiment for several inters with portable fences to establish the ost effective fence arrangement.

Because few rights-of-way are wide enough) permit the proper placement of snow fences, ermission is obtained to place the fences on private property upwind from the highway. When the private land is in cultivation the fences must be taken down each spring and re-erected in the fall, involving considerable expense for labor, breakage, and storage. Some States have therefore acquired additional right-of-way to accommodate windbreaks or living snow fences of trees and shrubs. Railroads in North America and Europe have used this form of protection for over 60 years in certain localities.

There is a considerable body of literature on the spacing, cultivation, and care of a variety of trees suitable for shelterbelts, windbreaks and snow hedges (*3, 9, 10, 11, 12, 13, 14*). Trees that retain a fairly high density during winter, such as evergreens, are generally best for snow hedges, but certain

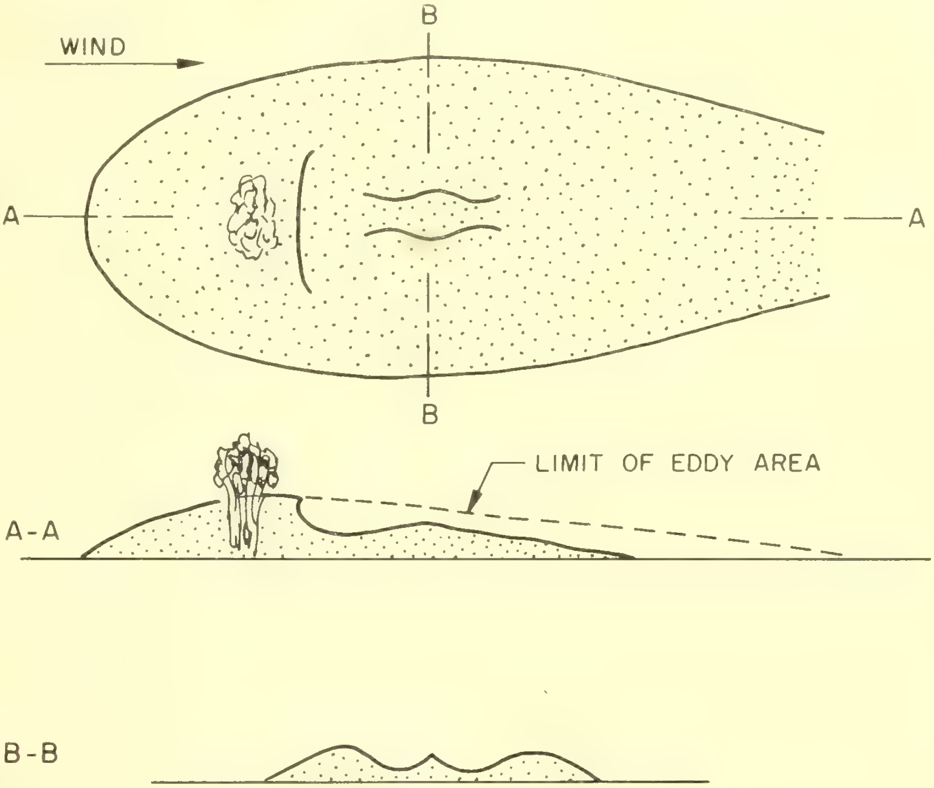

Figure 11.—Snow deposits caused by clump of bushes (Cornish, 1902).

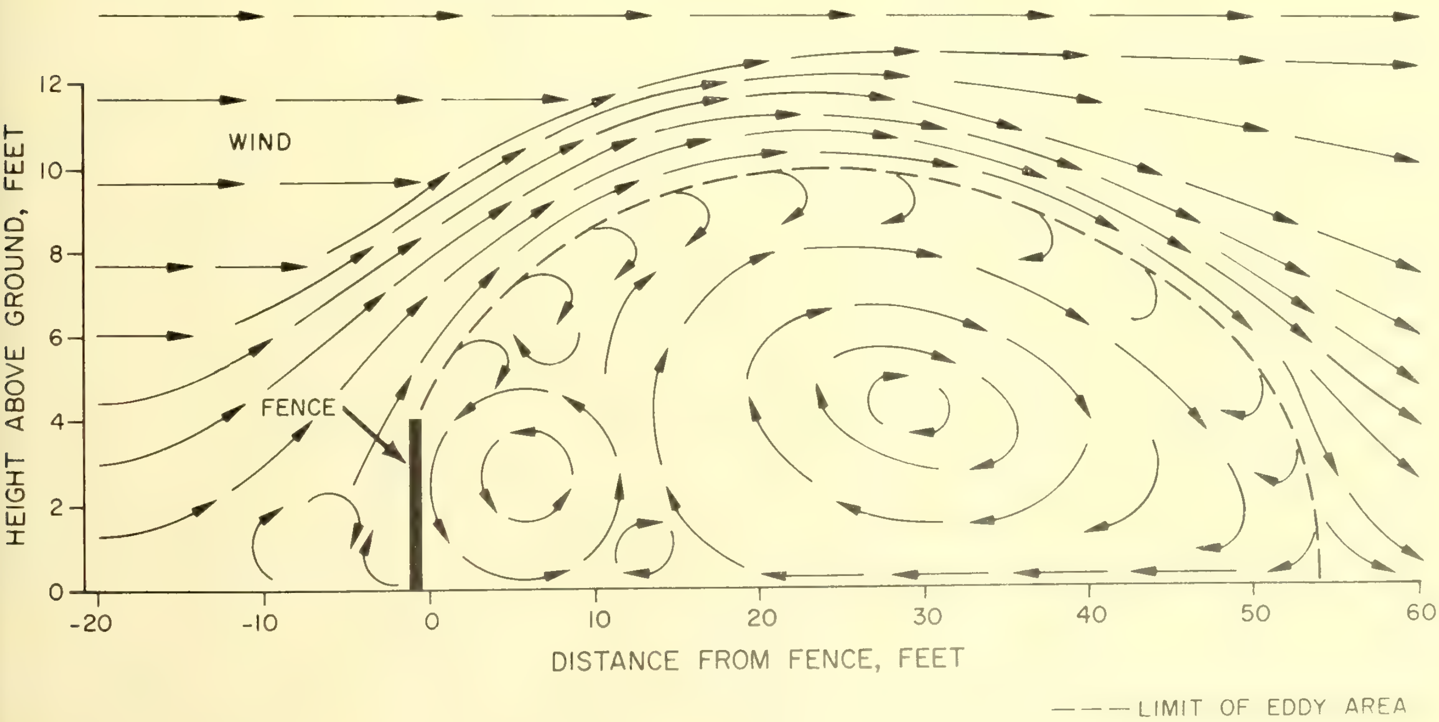

Figure 12.—Cross section of eddy area behind a solid fence (Finney, 1934).

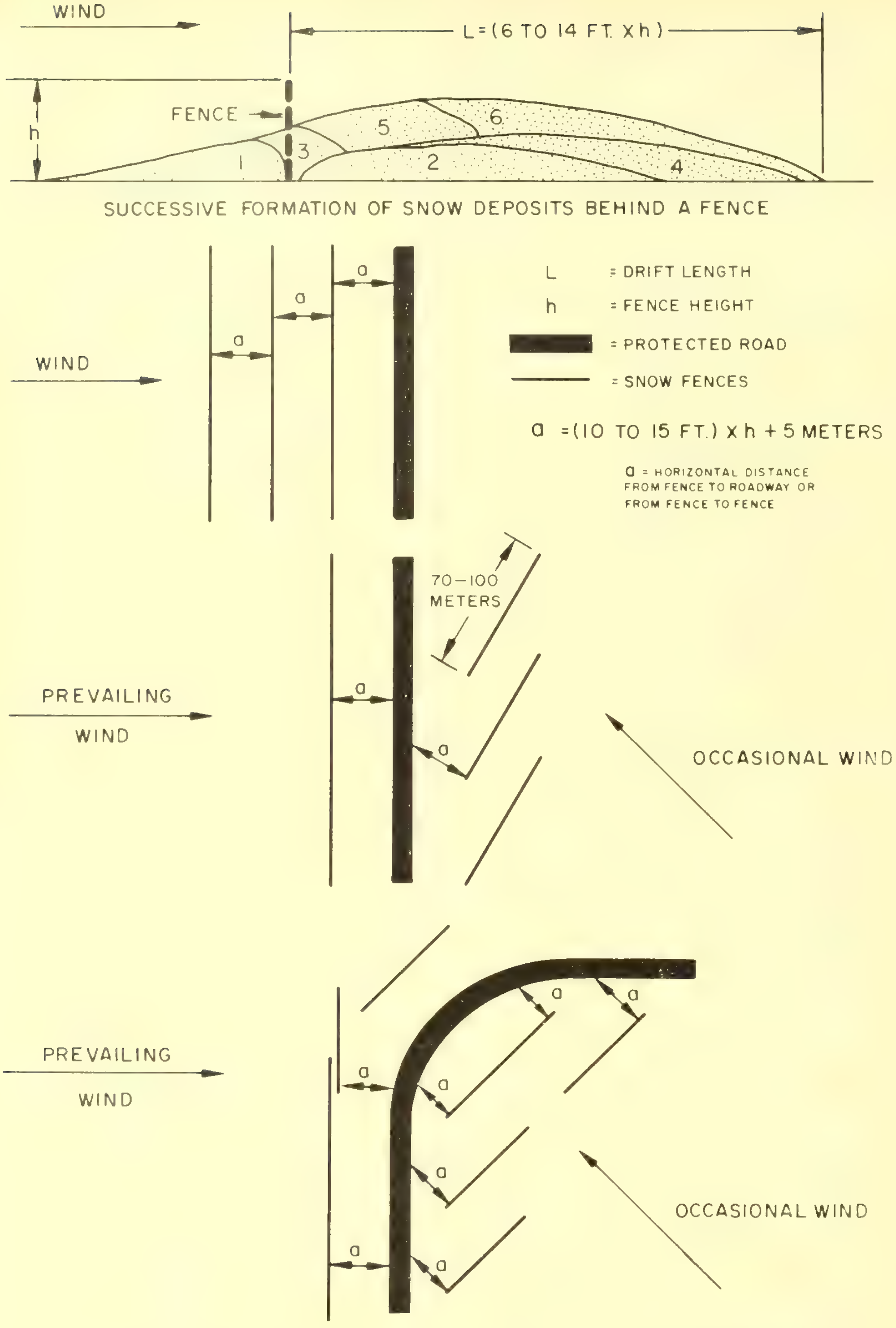

Figure 13.—Snow fence arrangements used in Germany (Bekker, 1951).

deciduous varieties are also useful (*1*). Snow accumulation is most effective when one or two dense shrub rows or branched small trees are used (*9*). Snow control hedges and shelterbelts also afford protection to birds and small game.

To be effective for snow control trees and shrubs do not need to be planted in long parallel rows. Discrete clumps, if properly planned, will work just as well, and with a more natural overall appearance (*1*). Stiffness and monotony of the plantings can also be relieved by varying the species, arrangement, and spacing (*14*).

Planting of hedges should be preceded by a number of years of operation with portable fences to establish by trial and error the best location and orientation for the permanent windbreak (*2*), and until planted hedges reach full effectiveness the drift spots must be protected by snow fences. Also, the snow hedges require annual care to maintain their effectiveness, but the cost is only about one-third as much as the annual re-erection of wooden snow fences. The effectiveness of the hedge or windbreak increases with age as the plants become higher provided that the density of the lower parts is maintained (*2*).

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Horizons in Highway Maintenance

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by [1] WILLIAM N. RECORDS
Highway Research Engineer,
Structures and Applied Mechanics Division

New horizons have opened up as a result of research and development studies. These studies provide the knowledge and techniques needed to make improvements in highway maintenance. The author describes the size of the maintenance function, explains the complexity and the problems that characterize highway maintenance, and emphasizes its challenge to the entire highway industry. He is convinced that, if all segments of the highway industry—management, research, design, construction, materials production, and equipment manufacture—rise to meet this challenge, substantial improvements can be made in maintenance during the next two decades.

MANY highway departments have attempted to reduce maintenance on ghways built during recent years by improving design practices, construction proceures, and materials. These efforts are ommendable, but total maintenance workads of new highways have increased steadily. he reason for this increase is that the reducons in workloads as a result of better design, onstruction, and materials have been more an offset by increases arising from more afety features, beautification, higher standds, and expanded services for highway users. hese same factors have caused substantial creases in the maintenance workloads for der highways. In addition, many older roads ere not built to carry their present traffic ad. Consequently, extensive work has been quired to avert failure in structural omponents.

Maintenance is a major function of highway epartments regardless of their size or area f responsibility. Indications are that it will ontinue to be a major function despite dvances in design, construction, and materials. It is logical, therefore, that maintenance should receive considerable attention om all segments of the highway industry.

Highway maintenance is characterized rimarily by (1) its size, (2) its complexity, (3) s many interrelated problems, and (4) its pportunities for improvement. Each of these spects is examined in some detail in the ollowing paragraphs.

Size of the Maintenance Function

There are more than 3.5 million miles of ighway in the United States and, at the resent time, about 30,000 separate organiations responsible for maintaining this vast etwork. These organizations employ over 00,000 men and use approximately 400,000 nits of major equipment in carrying out the ital maintenance function. Expenditures are n a comparable scale. Figure 1 shows reorted expenditures for highway maintenance y all governmental agencies during the period 950 to 1965. In 1965, the cost of highway aintenance reached $3.23 billion, which was early 23 percent of total highway expendiures in that year. Figure 1 also illustrates hat maintenance expenditures have been teadily increasing over a long period of time.

The increase was $1.81 billion or 127 percent during the 16-year period from 1950 to 1965.

In figure 2, expenditures have been roughly adjusted to a standard dollar by applying the Bureau of Public Roads Maintenance Cost Index. The adjusted values still show a significant increase, $0.61 billion or 30 percent, from 1950 to 1965. Part of this increase is attributable to new highways added to systems; the remainder reflects additional work generated by more safety features, beautification, higher standards, and expanded services.

Complexity of Highway Maintenance

Highway maintenance is a function of great complexity. Obviously, this fact is not widely appreciated in the highway industry. Many individuals—even those directly connected with it—tend to think of maintenance in terms of a few operations such as patching, mowing, removing litter, and plowing snow. These particular activities do account for substantial portions of the total workload, but they do not begin to encompass the total maintenance function. Research studies conducted by the Bureau of Public Roads have identified more than 100 distinct maintenance operations. Some of these are illustrated in figure 3. In addition, there are at least 30 operations that arise out of secondary functions assigned to most of the organizations responsible for maintenance. A single management field unit, such as a gang, may perform more than 50 of these operations each year. Table 1 shows an actual case which is considered typical.

The complexities involved when a single maintenance gang tries to perform so many different activities with any degree of effectiveness and efficiency should be considered. The picture is further complicated by the fact that most operations are performed (1) by several different work crews, (2) on many different occasions, (3) at many widely scattered worksites, and (4) under varying worksite conditions. Each time a crew sets out to perform a particular operation, they are, in effect, faced with a new situation. Results are influenced by many factors including weather, traffic, quality of supervision, training of personnel, size of the job, and the availability of materials, equipment, and manpower.

It is not surprising that research studies have uncovered substantial variations in the total quantity of work, quality of workmanship, productivity, and unit costs for the same operation, as performed by different maintenance crews or even the same crews on different occasions. Figures 4 and 5 are typical of the pattern observed in several maintenance organizations. It is surprising that maintenance organizations manage to do a reasonably acceptable job most of the time. This can be attributed largely to the devotion and ingenuity of individual engineers, managers, and employees.

Highway Maintenance Problems

Highway maintenance is a function characterized by many problems. These usually fall in two categories: (1) Management problems, and (2) technological problems. Management problems are those that arise from the process of planning, organizing, directing, and controlling the maintenance function. Technological problems arise out of the procurement and the use of facilities, equipment, tools, and materials.

A realistic appraisal of any maintenance organization will disclose that it faces many problems in both categories. Some of the problems uncovered by such appraisals are, as follows:

Management Problems

- Ineffective or nonexistent planning
- Budgeting based on inadequate factual data
- Reporting system that does not provide needed data

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[1] Presented at the Louisiana State University Highway Engineering Conference at Baton Rouge, La., Mar. 1, 1967.

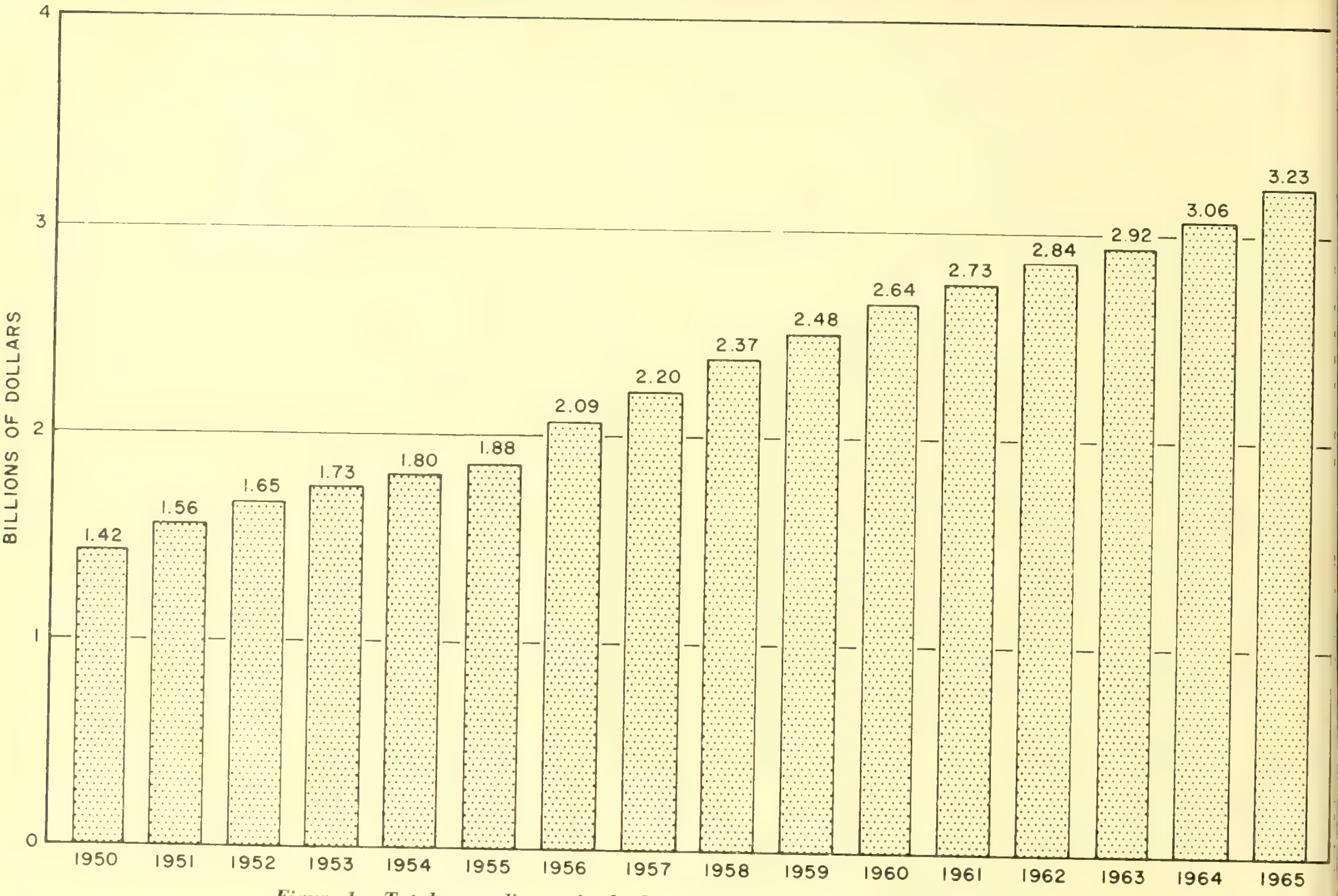

Figure 1.—Total expenditures for highway maintenance in the United States.[1]

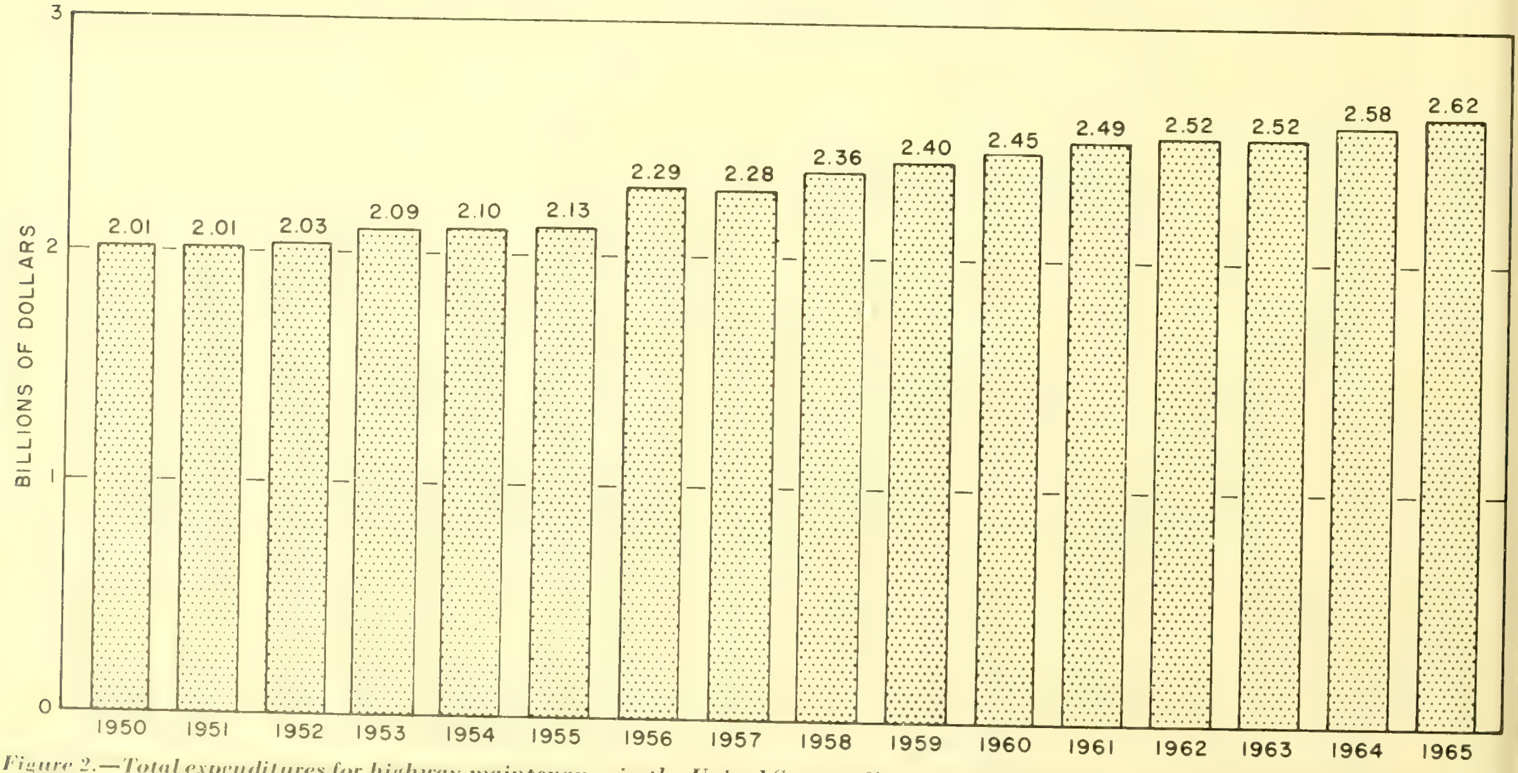

Figure 2.—Total expenditures for highway maintenance in the United States adjusted by Bureau of Public Roads Maintenance Cost Index (1957–1959 base).[2]

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[1] Data from Highway Statistics, Summary to 1965, U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, March 1967.

[2] Basic data from Highway Statistics, Summary to 1965, U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, March 1967.

- Poor communications
- Central office staff too small
- District staff too small
- Poorly trained field supervisors
- Widely varying productivity and unit costs
- No standardized work methods
- No definitive standards
- No formalized process for evaluating performance

Technological Problems

- Insufficient facilities
- Poorly designed or obsolete facilities
- Insufficient equipment
- Equipment not well suited for work to be done
- Inadequate equipment maintenance
- Materials not well suited for work to be done

Of course, no single organization will be faced with all these problems, nor will a particular problem be of equal severity for two organizations. But it is recognized that all maintenance organizations have problems and some of these are relatively severe.

Another aspect of the problems facing maintenance organizations should be recognized. They are often interrelated. For example, the problem of insufficient equipment is nearly always interrelated with problems in planning, budgeting, or work methods. Because of these interrelationships, effective solutions usually cannot be developed on a single-problem basis. Some attempts to solve a single problem have failed because the solutions were detrimental to the total maintenance function. This experience has led to the concept that maintenance problems can be solved best by an approach that deals with them both individually and totally.

Opportunities for Improvement of Highway Maintenance

Highway maintenance—large and complex—can be improved. New horizons have opened up as a result of research and development studies which are being conducted in Louisiana, North Carolina, Virginia, Utah, and a few other States. These studies are providing the knowledge and techniques needed to make improvements. They also serve to point out clearly the challenge that maintenance offers to the entire highway industry. If, after the next 20 years, the industry has risen to meet this challenge and has reached out for these new horizons, it is possible to visualize a topflight maintenance operation in 1987 that is capable of functioning as follows:

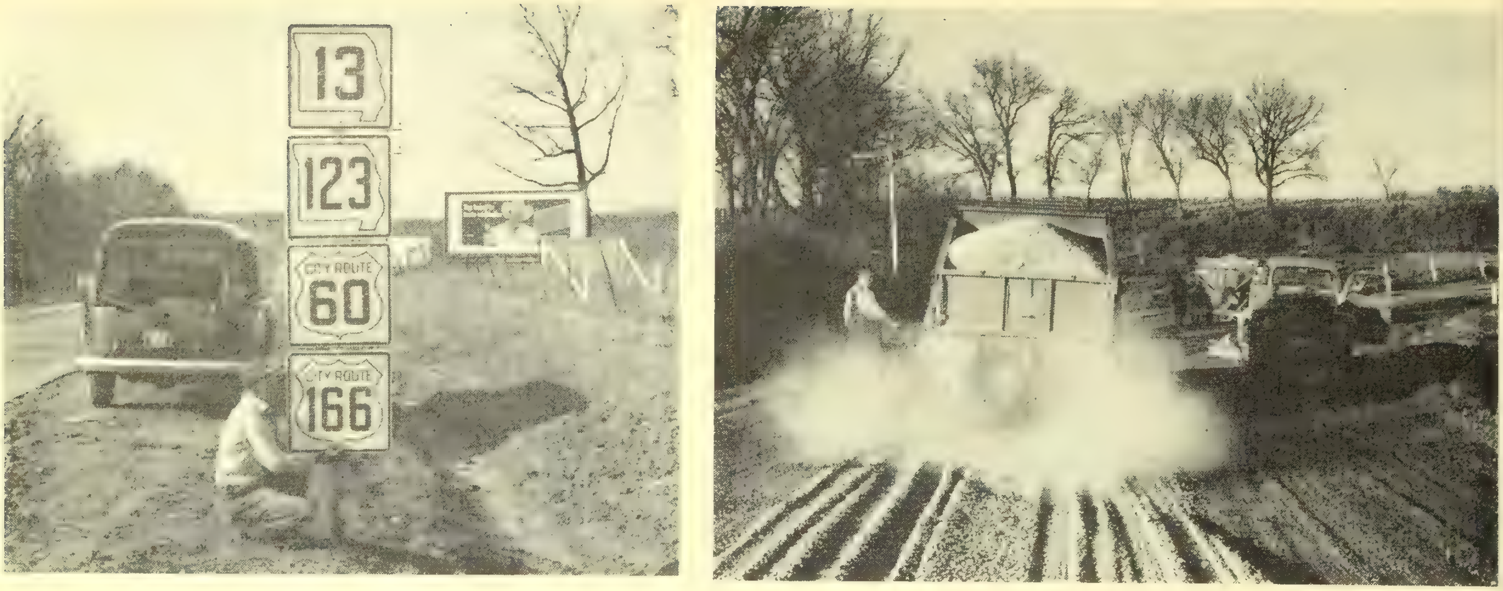
- It is manned by career engineers, managers, and technicians who are highly skilled in the use of scientific methods, automated equipment, and specialized materials.
- It is equipped with modern facilities and equipment designed expressly for maintenance.
- It has a communication system capable of providing instantaneous contact with practically every employee.

Table 1.—Distribution of labor time in Johnson County

[Including distributed overhead]

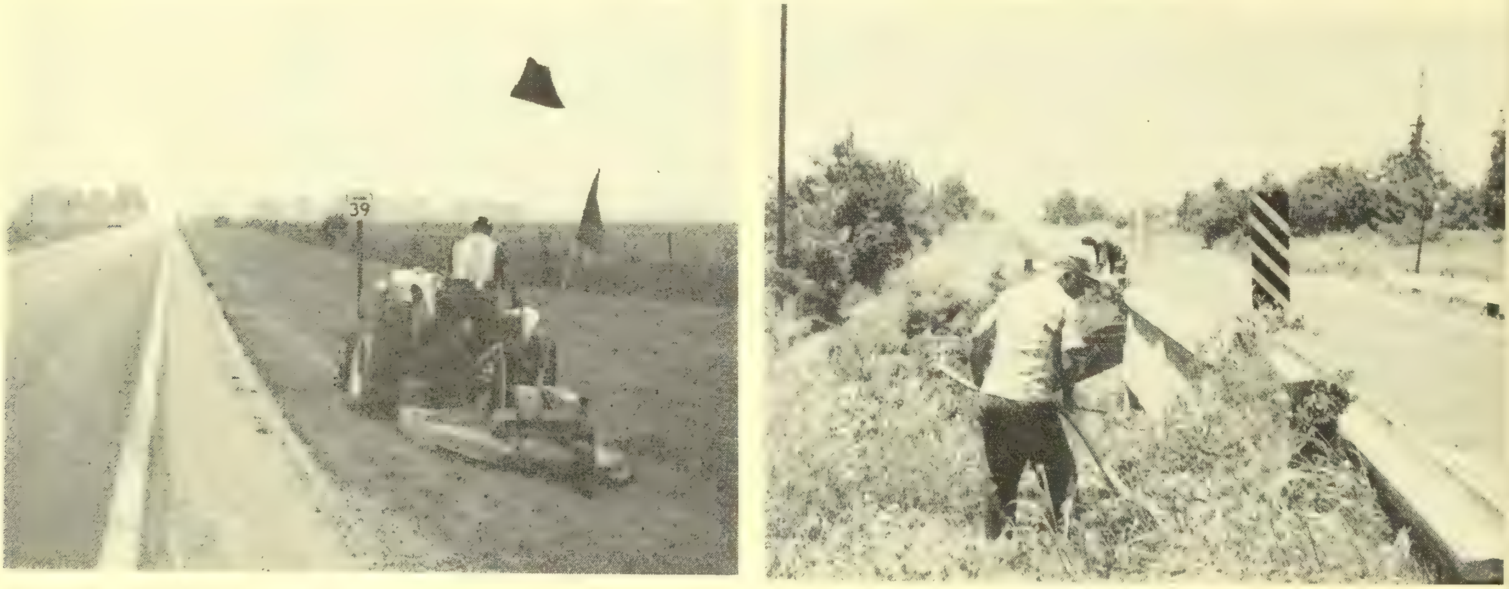
Operation	Labor time		Proportion of total available working time	
	Per operation	Total	Per operation	Total
OVERHEAD OPERATIONS—UNDISTRIBUTED	*Hours*	*Hours*	*Percent*	*Percent*
Supervise maintenance activities		1,976		6.0
Service and repair equipment		6,019		18.3
Clean, repair, or improve garage facilities		1,211		3.7
DIRECT OPERATIONS—MAINTENANCE OF EXISTING SYSTEM				
Routine surface:				
Patch roadway surfaces with aggregate	1,481		4.5	
Patch roadway surfaces with bituminous cold mix	2,095		6.4	
Blade gravel surfaces	286		0.9	
Fill joints and cracks in roadway surfaces	177		0.6	
Apply dust palliatives to gravel surfaces	142		0.4	
Clean or drain roadway surfaces	34		0.1	
Total		4,215		12.9
Special surface:				
Mudjack concrete pavements	303		0.9	
Seal bituminous and concrete pavements	1,662		5.1	
Resurface with bituminous mixes	758		2.3	
Plane or roll bituminous pavements	136		0.4	
Total		2,859		8.7
Shoulder and approach:				
Patch shoulders and approaches with soil	589		1.8	
Patch shoulders and approaches with aggregate	791		2.4	
Patch shoulders and approaches with bituminous cold mix	917		2.8	
Reseed or resod shoulders and approaches	102		0.3	
Blade or reshape shoulders and approaches	226		0.7	
Total		2,625		8.0
Roadside and drainage:				
Repair cut- and fill-slopes	177		0.5	
Repair or replace pipes and tiles	87		0.3	
Clean pipes, tiles, and box culverts	257		0.8	
Clean or repair unpaved drainage ditches	430		1.3	
Clean paved flumes, gutters, and drop inlets	4			
Remove trees from roadsides	89		0.3	
Mow roadsides with tractors (including shoulders)	1,596		4.8	
Spray weeds on roadsides	157		0.5	
Total		2,797		8.5
Snow and ice:				
Remove snow from roadway surfaces and shoulders	3,780		11.5	
Erect snow fences	619		1.9	
Remove snow fences	492		1.5	
Sand roadway surfaces	469		1.4	
Salt roadway surfaces	691		2.1	
Remove ice from roadway surfaces and shoulders	485		1.5	
Remove snow and ice from drainage ditches	89		0.3	
Put out and remove cinder barrels	122		0.4	
Total		6,747		20.6
Traffic service:				
Paint centerlines and edgelines	479		1.5	
Paint bridge endwalls, medians, and miscellaneous markings	317		1.0	
Erect, replace, repair, or paint signs and guideposts	606		1.8	
Clean signs and reflectors	97		0.3	
Remove or paint guardrails	77		0.2	
Miscellaneous work at weigh stations and roadside parks	88		0.3	
Total		1,664		5.1
Other:				
Clean or repair bridges	29		0.1	
Remove litter from right-of-way	140		0.4	
Miscellaneous work resulting from disasters	15		0.1	
Miscellaneous work resulting from maintenance contracts	48		0.1	
Total		232		0.7
DIRECT OPERATIONS—NEW CONSTRUCTION				
Detour:				
Patch roadway surfaces with bituminous cold mix	174		0.5	
Rebuild aggregate base courses	168		0.5	
Mow roadsides with tractor (including shoulders)	19		0.1	
Erect, replace, repair, or paint signs and guideposts	244		0.8	
Miscellaneous work	142		0.4	
Total		747		2.3
Miscellaneous work resulting from construction contracts		111		0.3
Net available working time		31,203		95.1
NONOPERATIONAL				
Major nonoperational delays		1,597		4.9
Total available working time		32,800		100.0

[1] Source HRB Special Report 65, Supplement I.



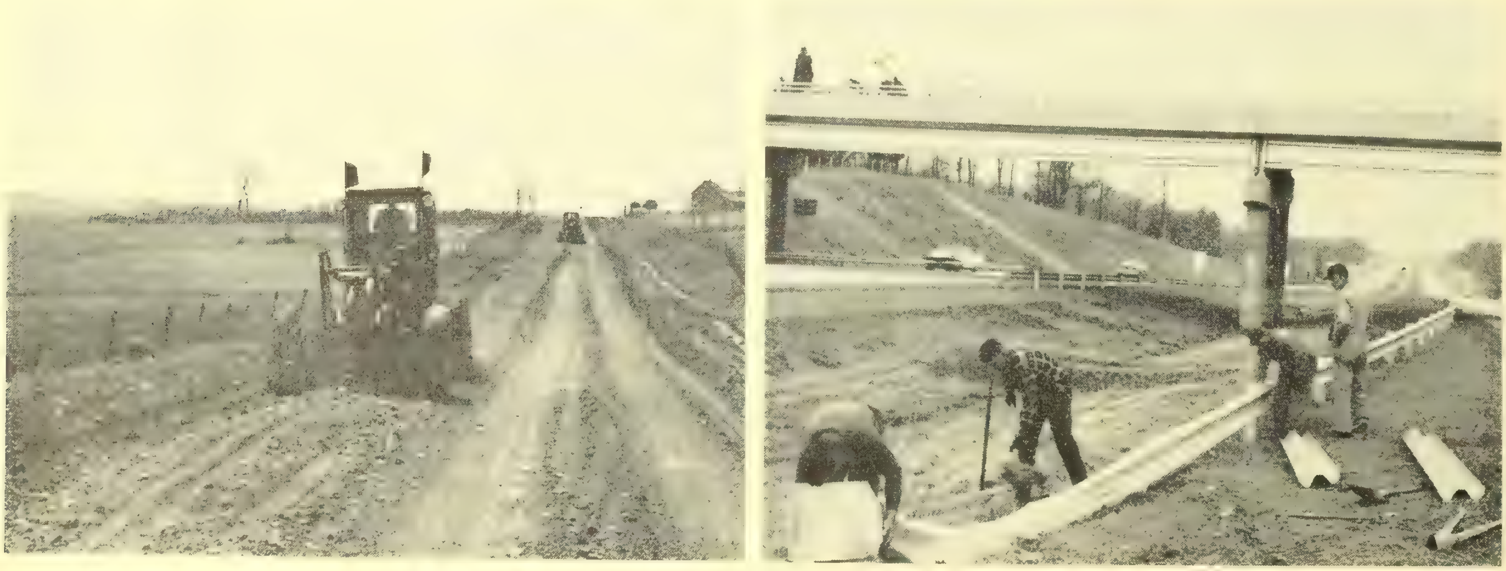
Replacing sign

Regraveling surface



Tractor mowing

Hand mowing



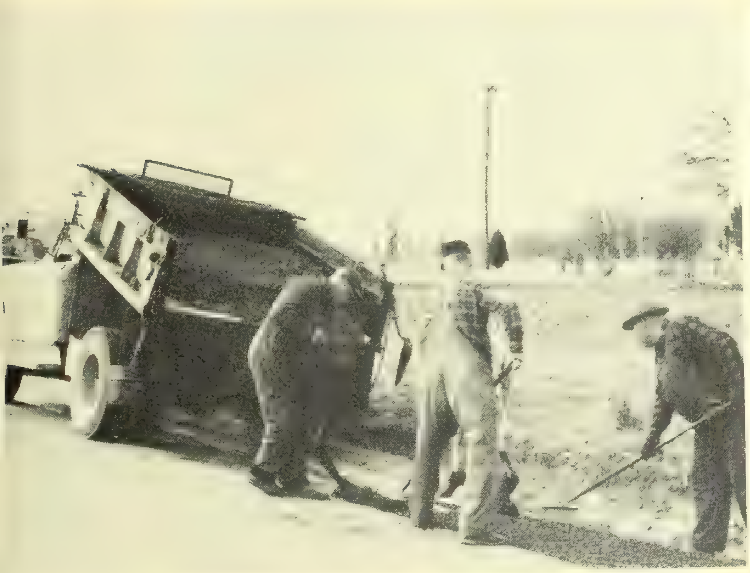
Blading gravel surface

Replacing damaged guardrail

Figure 3.—Typical maintenance operations.

Picking up litter

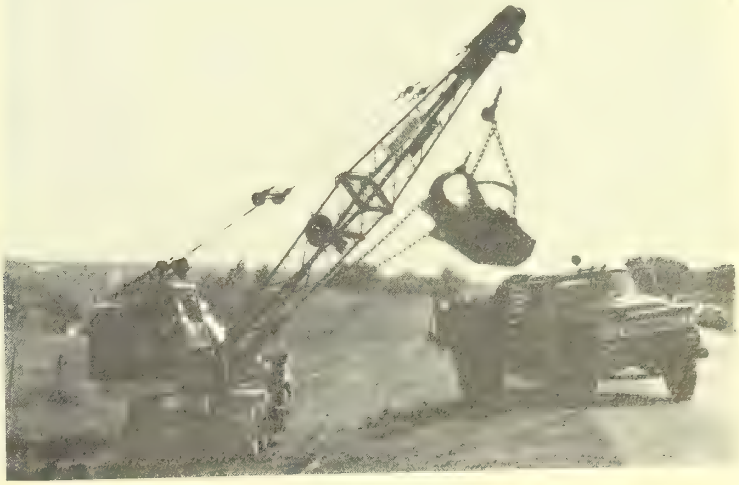
Painting pavement marking



Filling shoulder edge rut

Patching pavement

Plowing snow



Cleaning ditch

Figure 3.—Typical maintenance operations (continued).

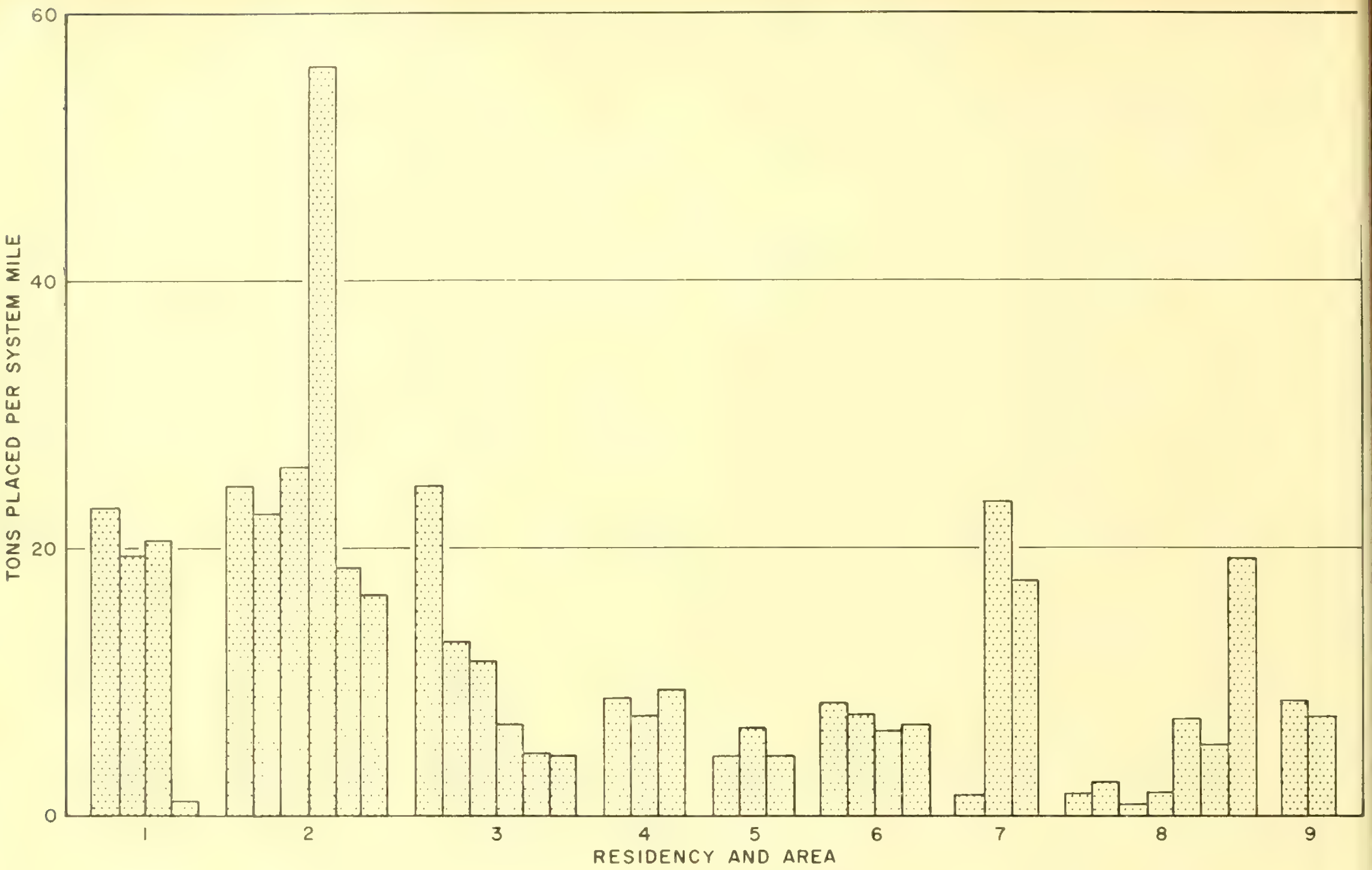


Figure 4.—Average amount of skin patching on surface treated secondary roads.[3]

- A wide spectrum of data is recorded automatically or semiautomatically and transmitted directly to computers for processing.
- There is a strong and continuing program to seek out and develop new methods, equipment, and materials.
- There is a strong and continuing program to train all personnel using sophisticated techniques and advanced equipment such as programed instruction and closed circuit television.
- New types of nondestructive testing equipment are used to detect incipient failures in structural components. (Many components of highways require practically no maintenance at all other than replacement or rehabilitation at long intervals.)
- Many failing or damaged structural components are replaced by standard, preassembled units.
- Failing pavements are patched with a liquid material that is self-leveling and hardens almost instantly.
- At regular intervals, ranging from 1 to 6 weeks, every highway is traversed by crews which simultaneously remedy all known defects and take care of all needed services.
- Highways are continuously patrolled by maintenance personnel who make inspections, take care of emergency situations and aid motorist in trouble.
- Expenditures for maintenance are, relatively speaking, no larger than those at the present time.

These horizons can be reached if the highway industry is willing to devote the necessary attention and effort toward improving maintenance during the next two decades. Highway construction operations offer ample proof that the industry is capable of making significant improvements. One good example is portland cement concrete paving. In 1947, a typical paving crew consisting of 40–50 men could produce and place about 50 cubic yards of concrete per hour, using a single dual-drum paver. Today, many crews of only 30–40 men use a central mix plant and a slip form paver to produce and place about 300 cubic yards per hour. This spectacular improvement has been the result of intensive effort by several segments of the highway industry.

The information available indicates that personnel of the highway industry already have the know-how to effect improvements which would reduce total maintenance expenditures 5–10 percent without sacrificing anything in quality or level of service. Nationally, this could mean savings of over $150 million per year. The savings could be used to improve safety, raise standards, maintain new highways, or construct more new highways. This is not a one-time saving; it would be repeated each year. Ultimately, it should be possible to effect improvements that would result in much larger savings—large enough, in fact, to more than cover the cost of transforming existing maintenance organizations and operations into something akin to what is imagined for the future.

To move forward toward these new horizons, there must be a change in attitude on the part of all segments of the highway industry. Interest must be stimulated, complacency converted into dissatisfaction with the status quo, and lethargy changed into a willingness to take action. Once the right climate is created, it will not be difficult to move into an action stage where problems can be identified and defined, the needed technology can be developed, and improvements can be instituted. If all segments of the highway industry—management, research, design, construction, materials production, and equipment manufacture—participate fully, the maintenance function will be improved and these new horizons can be reached.

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[3] Data from Virginia Maintenance Study Report—Part II, Performing Highway Maintenance Operations in Virginia, by Virginia Department of Highways, January 1966.

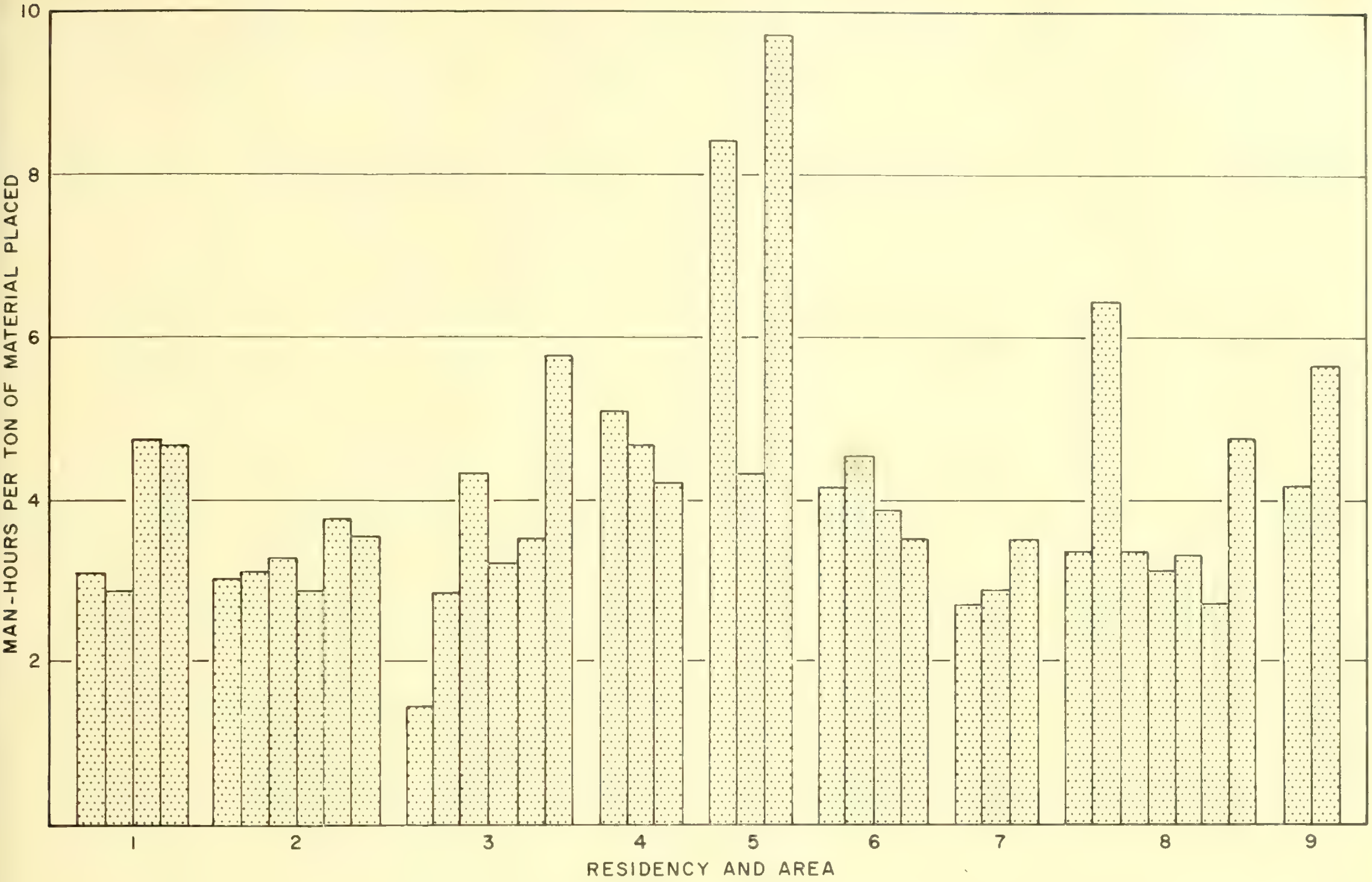

Figure 5.—Average productivity on skin patching.[3]

NEW PUBLICATIONS

The Bureau of Public Roads has recently published four documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Guidelines for Trip Generation Analysis

Guidelines for Trip Generation Analysis (65 cents a copy), published in June 1967, is one of a series of procedural manuals and guidelines produced by the Office of Planning in an effort to document the various elements of the transportation planning process.

Trip generation is commonly used to describe the number of trips starting or ending in a particular area in relation to the land use or socioeconomic characteristics of that area. These guidelines provide a framework for the conduct and evaluation of the trip generation element of the planning process. Existing techniques are documented and new approaches and emerging concepts in trip generation analysis are discussed.

Overtaking and Passing on Two-Lane Highways

Overtaking and Passing on Two-Lane Highways (20 cents a copy), a 44-page research and development report, contains a review of the published literature reporting observational and experimental studies of overtaking and passing maneuvers on highways having only 2 lanes. The literature review was conducted for the Bureau of Public Roads as part of a contract directed toward gaining a conceptualization of overtaking and passing maneuvers. The work performed by the contractor was in support of the Public Roads research and development program to reduce the number of rear-end and head-on accidents on rural highways and to increase the service volume on them.

As most of the work reviewed was performed in the 1930's or early 1940's, and the emphasis was on providing the highway engineer with design information for laying out passing zones, much of the data presented is concerned with the times and distances required to pass; relatively little information is presented on passing judgment—the conditions under which a driver will or will not pass.

To facilitate comparisons between the results of the different studies, the review is organized topically as follows: Distance required to pass, time required to pass, passing headway, passing performance, passing reaction time, traffic conditions and passing, and gap acceptance.

Typical Plans for Retaining Walls

Typical Plans for Retaining Walls, 1967 (45 cents a copy), are intended to serve as a useful guide to State, county, and local highway departments in the development of suitable and practical retaining wall designs and should be particularly valuable to the smaller highway departments with limited engineering staffs. An effort has been made to give sufficient information on all plans so that they may be readily adopted in the preparation of contract drawings.

(*Continued on p. 250*)

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[3] Data from Virginia Maintenance Study Report, Part II, Performing Highway Maintenance Operations in Virginia, by Virginia Department of Highways, January 1966.

Residential Density Structure—An Analysis and Forecast With Evaluation

BY THE
OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by[1] CARL N. SWERDLOFF,
Highway Research Engineer,
Urban Planning Division

In this article Swerdloff reports on an analysis of the residential density structure of a small urban area. The procedures discussed for studying known density structures and forecasting future ones are simple, and can be readily used by planners with minimal data resources.

Such a study is valuable to the urban transportation planner concerned with bringing order and efficiency to urban space. Through the development of urban density structure models that forecast future population density distribution and the likely fluctuations in residential development caused by changes in planning standards or socioeconomic conditions, the transportation planner can favorably influence future residential density patterns. By providing and depriving transportation access the planner establishes heterogeneity in the distribution of land value and consequently in the pattern of population density. Because transportation accessibility is an effective control on residential population density distribution, the location of future transportation facilities influence the location and intensity of residential development.

Introduction

URBAN population density has been of interest to professionals in many different, but related disciplines. Since 1951 when Colin Clark (*1*)[2] published his exponential decay formulation of the spatial arrangement of urban population densities, economists ecologists, geographers, city planners, and others have been concerned with determining the universality of the Clark formulation. The factors contributing to interregional variability in the parameters and in the temporal stability or lack of stability of the formula are still questioned.

The onrush of urban transportation planning studies in the late 1950's focused attention on the fact that the future land-use distribution in urban areas must be estimated before transportation demands could be forecasted. Attention to the analysis and forecasting of urban land use necessarily involved an increased interest in the matter of population density. The measurement of population or residential density is a necessary part of the land-use distribution process used in traditional transportation planning programs. The population density estimates are used either to determine whether the forecasted population distribution shows realistic zonal residential densities, or to compute the consumption of previously vacant land by the increment of residential growth in each zone. The computation of consumption of previously vacant land is of particular importance when the urban simulation process is performed in a finite number of consecutive time periods that require the relevant data to be updated at the conclusion of each time period in preparation for the next time period. The residential population density is also used in the population distribution process as illustrated by the individual density submodels used in the forecasting model system developed at the Delaware Valley Regional Planning Commission (DVRPC) (*2*). In the DVRPC simulation system the land-use distribution reflects the existing residential density pattern of the study, just as the forecast residential density pattern reflects the existing land-use distribution. However, the land-use distribution processes usually rely on exogenously determined land-use density values as explanatory variables.

Land-use density is used in trip generation analysis in many transportation studies. Trip generation—the estimation of the total number of trips originating or terminating in each analysis zone—is frequently determined by use of multiple regression analysis equating trip production with measurable characteristics of the analysis zone, such as existing density. Trip generation analysis thus shows that the arrangement of households will affect the volume of daily trips. The external economies associated with more dense land-use arrangements apparently influence trip generation. Transportation planners are currently interested in mode split that is, the distribution of daily travel between private and public transportation facilities. The land-use density pattern is important to this area of analysis; and speculation on the future role, function, form, and viability of large urban regions is contingent on the assessment of the density levels that the future population will be willing to maintain.

The lack of operational procedures and normative guides for estimating the future distribution of urban residential densities has prompted the study presented in this article. The residential density pattern of a small urban region was investigated in two time periods 12 years apart. The author has attempted to establish the conformity of the observed density structure of the study area to the universal formulation of urban population density by Clark (*1*). Many multiple regression analyses were undertaken and the observed shifts in the parameters over the 12-year study period are presented. Also, the author has attempted to examine the relative accuracy of each of the forecasts of the residential density structure. The analysis and forecasting procedures examined were kept simple so that they would be practical for application in transportation planning studies for moderate sized urban areas.

Residential density refers to the ratio of some measure of the volume of residential activity to the total land or space used. But there are many ways in which land use and total land can be defined and measured (*3*). The numerator of the ratio is the volume of residential activity; that is, the number of persons, households, or dwelling units. For the purposes of the study reported in this article it was expressed in total dwelling units. The denominator of the ratio, the total land or space area, has been a source of definitional inconsistency, but for the purposes of this paper gross density land area refers to the total area of the analysis unit; that is, the area arrived at by planimetering the boundaries of the land. Gross census tract dwelling unit density is then calculated by dividing the total number of dwelling units in a census tract by the total land area contained within the boundaries of the tract. Net residential density land area is a sharper measure than is gross density primarily because it reflects the actual uses of land that make up the total land area of the different analysis units. For example, all land constituting a given census tract can be classified as used or vacant and the net census tract dwelling unit density per square foot of used land computed. Used land can be further classified as residential or nonresidential and a dwelling unit density per square foot of residential land computed. For the study reported here net residential density is defined as total number of dwelling units per unit of residentially used land, including street area. Because of personal bias, the bulk of the analysis was in terms of net density.

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[1] Presented at the 46th annual meeting of the Highway Research Board, Washington, D.C., Jan. 1967.

[2] References identified by italic numbers in parentheses are listed on page 250.

Conclusions

Results from the study reported in this article are not conclusive because they are based only on a single analysis of a single urban area over a single time period. It is the belief of the author, however, that the procedures for the analysis and forecasting of residential density in small and medium size urban areas can be advanced through the results of the study. Several conclusions have been drawn from the study results.

- The author believes that future analysis of urban residential structure should be concentrated on net as opposed to gross density measures. Net density is a more exact measure, is compatible with a substantial body of existing theory, and is possibly more conducive to meaningful analysis and projection.

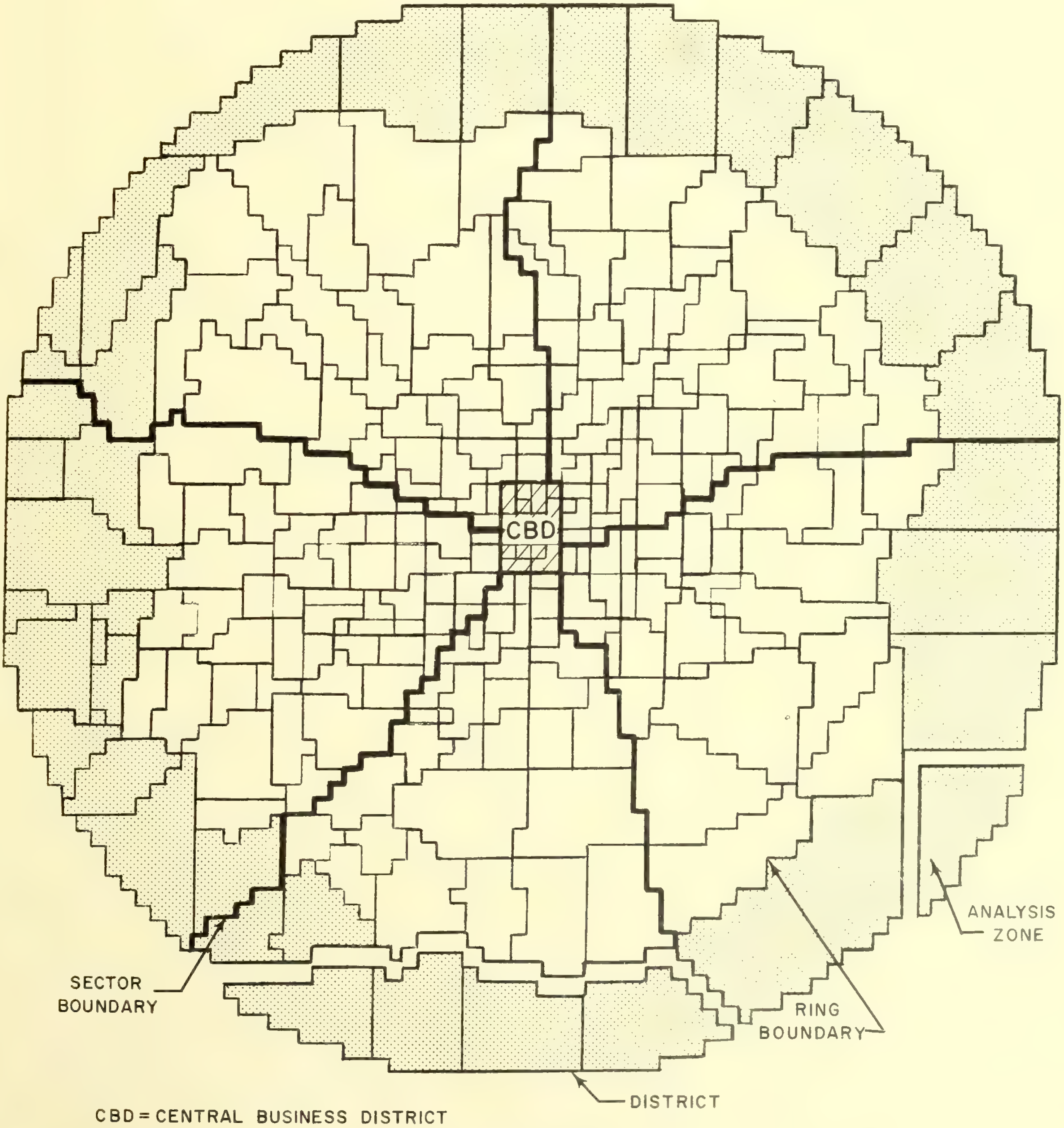

Figure 1.—Greensboro, North Carolina Study Region.

- The district unit of analysis used in the study reported here has an average internal population of 2,000 persons and is comparable in size to the urban transportation travel analysis zone. It is a useful level of aggregation for the study of urban residential density structure.

- Density distance gradients are useful in analyzing the density structure of the urban area and can also serve as forecasting devices. However, because density gradients are not static, the author suggests that additional research be devoted to the development of rational explanations and procedures for estimating the parametric shifts so that the forecasting potential of density distance gradients can be improved.

- Accurate net residential density models that are structured in terms of existing theories of economic equilibrium and land-use distribution should be developed.

- Central core areas should be separated from the rest of the urban region in the development of residential density models. Also, the inclusion of substantially rural areas, in which significant urbanization is not expected, distorts residential density analysis.

- Future residential density configurations should not be used merely as exogenously determined input constraints for residential location simulation models. Future residential densities should and can be functionally related to a number of socioeconomic variables, including the existing and forecasted urban activity location pattern.

Study Conditions

The Greensboro, N.C., metropolitan region was the locale for the study reported here. A complete data supply for Greensboro was available at a detailed geographic level for the years 1948 and 1960 (*4*, *5*). The primary data contained measures of total dwelling units, land area for usable and unusable land, and assessed land value, which were coded to 3,980 square foot grid cells covering the circular study area, approximately 8 miles in radius, centered around the Greensboro central business district (CBD). The data also contained measures of the proximity to urban activities and CBD. Additional data on total employment and an index of accessibility to the employment was also available at the travel analysis zone level. Figure 1 shows the entire study area structured into 249 analysis zones and five sectors, and illustrates the form of the analysis rings and districts. Average family income was not present in the original data, but was available for 1950 and 1960 from the U.S. Bureau of the Census publications (*6*, *7*). Each zone was assumed to have an average family income equal to the mean for the census tract into which it fell.

The study area was divided further into five sectors radiating out from the center of Greensboro and into circular rings 1-mile wide. The sector boundaries of the study area were forced to analyses zone boundaries. The first ring was one-half mile in radius and circumscribed the central core of Greensboro City.

Table 1.—Analysis of variance results for interdistrict net residential density

Source of variation	Sum of squares	Degrees of freedom	Estimate variance	F
Between districts	6, 860. 1	39	175. 9	[1] 2. 92
Within districts	11, 494. 4	191	60. 2	------

[1] $F=\frac{175.9}{60.2}=2.92$ significant at 0.001 level.

Table 2.—Analysis of variance results for intersector and interring net density

Source of variation	Sum of squares	Degrees of freedom	Estimated variance	F
Between rings	703. 1	7	100. 4	[1] 29. 5
Between sectors	14. 8	4	3. 7	[2] 1. 1
Residual	98. 8	29	3. 4	--------

[1] $F=\frac{100.4}{3.4}=29.5$—significant at the 0.001 level.

[2] $F=\frac{3.7}{3.4}=1.1$—not significant.

The primary analysis unit utilized in this study was the district defined as that area contained within the intersection of successive sector and ring boundaries. The study area could have been structured into *driving time to the CBD* time increment rings as opposed to distance increment rings, but previous work (*4*) with the same data showed little advantage to either method of structuring. Therefore, distance units were selected primarily to correspond with the bulk of existing structural density analyses data. Excluding the central core ring, the remaining 8-mile rings and five sectors totaled 40 districts. The district was used as the basic analysis unit because it most nearly approximated, in average resident population, the typical traffic analysis zone used in traffic simulation analyses. The average district in Greensboro had a total of 660 dwelling units in 1948. The traffic analysis zone was previously shown to be too fine in residential location model tests (*4*) and was therefore judged inappropriate for net density analysis.

To justify the use of the district as the analysis unit, a one-way analysis of variance was performed on the 1948 net residential density. Results of the analysis are shown in table 1. Statistically significant between-district variance indicated that the district aggregation of zones did not destroy the prevailing zonal net density variability; therefore, the district was shown to be an appropriate analysis unit. The central core district and its composite analysis zones were removed from analysis in the study reported here because density quantities computed for the central core district consistently deviated substantially and illogically from what would be expected, based on the findings for the remainder of the study area. The central core area is traditionally distinct in residential terms and is often treated as such in transportation simulation analysis; also the quality of the residential land use and dwelling unit data is often less reliable than that for the remainder of the urban area and could cause further difficulty in analysis.

The sectors and rings were investigated as additional levels of geographic analysis of residential density structure. A two-way analysis of variance was performed on the 1948 zonal net density values stratified by ring and sector. Results of this analysis, shown in table 2, reveal that between-ring variability was significant but between-sector variability was not and the sector analysis unit was eliminated. Apparently distance away from the region's center is a more appropriate indicator of prevailing residential density than is distance in an angular direction from some reference axis. Having investigated several possibilities, the district and ring were determined to be the most appropriate geographic units for the analysis of the distribution of gross and net residential density. The central Greensboro core area was not considered in the analysis.

Methodology

The study reported here was an investigation of the residential density structure of a typical small-sized urban region. During the 12-year period from 1948 to 1960 the study area increased 52 percent in number of dwelling units, a rate of growth above the average for the Nation. Density-distance gradients were developed for both the 1948 and 1960 condition using the least squares criterion. Multiple regressions of net density were calibrated for 1948 and 1960; expected error in forecasting density was determined. The primary objectives of the study were (1) to investigate the appropriateness of several simple techniques that could be used by a small planning study with a minimal data supply; (2) to provide comparative quantitative measures of forecasting accuracy in procedures and methods investigated, and (3) to present any apparent advantage or disadvantage for selecting either gross or net residential density as the unit of measurement. Because only a small amount of material on the forecasting of trends in residential density exists and because the data were available for only two time periods, the selection of even simple forecasting techniques was severely limited.

Two district multiple regression analyses were formulated for the 1948 condition and were tested as valid forecasting devices. The initial regression formulation was modeled after the general form of the SPACEC I submodel of the Delaware Valley Regional Planning Commission Activities Model system. The second regression formulation was modeled in the usual multiple linear form. All calibration and forecasting errors are reported as coefficients of determination (R^2) and therefore maintain comparability for cross-comparisons:

$$R^2=\frac{\text{Original variance}-\text{explained variance}}{\text{Original variance}}$$

Where estimates by particular techniques are transformations of density, that is logarithmic, they have been converted to density prior to the computation of residual error.

Gross residential density has been defined as the number of dwelling units per unit of land. By using total land area and not land divided into units according to use, simplified gross density measures are achieved that give a superficial measure of the use of land by individual households. Gross density provides little improvement over a simple accounting of the fluctuations in dwelling units in a time series analysis and provides little information on the living compactness of the population. Gross residential density (D_G) is defined as:

$$D_G=\frac{\text{D.U.'s}}{A} \qquad (1)$$

Where

D.U.'s = Number of dwelling units in the analysis unit.
A = Land area of the analysis unit.

That is, a general expression of gross density for any geographic unit, i, may be expressed as:

$$D_{G_{it}}=(c)\ \text{D.U.'s}_{it} \qquad (2)$$

Where,

$D_{G_{it}}$ = Gross density in analysis unit i for time t.
D.U.'s$_{it}$ = Number of dwelling units in analysis unit i for time t.
c = Constant equal to $\frac{1}{A}$.

The gross residential density of each 8-mile-wide ring in the study area was computed for 1948 and 1960. Land-use data from a 1,000-foot square grid file of the total analysis land area was coded in ninths of land units according to the particular use of the land; therefore all residential density values are in terms of dwelling units per ninth of the total land area of the 1,000-foot square grid.

This rather awkward dimension does not affect the structural analyses or the measures of calibration and forecasting accuracy. Any of the density values reported in this paper can be converted either to dwelling units (D.U.) per acre by multiplying by the constant 0.392 or to D.U. per square mile by multiplying by the constant 250.9. The 1948 gross residential density results were plotted as a function of the distance of the ring from the CBD, in miles. A nonlinear relation exists between gross density and distance from CBD. A logarithmic plotting of the gross density data is shown in figure 2. A straight line fit in figure 2 would give evidence of a negative exponential relation of gross density to distance from CBD, but a definite straight line tendency was observed. A simple linear regression line fit to the gross density points resulted in the following equation:

$$ln\ D_{G\ 48}=2.43-0.648X \qquad (3)$$

Where,

X = Miles from the CBD.
$D_{G.48}$ = Gross residential density 1948.

When this regression equation is transformed to exponential form, the result is:

$$D_{G.48}=11.36e^{-0.648X} \qquad (4)$$

The least squares fit obtained for the dependent variable in logarithmic form will not necessarily result in the best equation in terms of minimum residual variance when the equation is solved for the dependent variable in nonlogarithmic terms.

The coefficient of determination (R^2) for equation 4 was computed to be 0.886. The same equation, calibrated on the ring gross density values, was examined when it was applied to the gross density measures at the district level. Solutions to equation 4 for the districts yielded an R^2 of 0.834. A reduced R^2 is to be expected because of the disaggregation and the resultant introduction of greater variability.

When the ring gross densities for 1960 were plotted, the nonlinear relation was similar to that found for the 1948 data. Semilogarithmic plotting of the 1960 points, presented in figure 3, resulted in the following least squares equation:

$$ln\ D_{G.60}=2.75-0.585X \qquad (5)$$

which transforms to:

$$D_{G.60}=15.58e^{-0.585X} \qquad (6)$$

The R^2's for equation 6 for 1960 ring analysis and district analysis are 0.989 and 0.923, respectively. The marginal shifting of parameters between the 1948 and 1960 gradients suggested that the 1948 equation be tested as a predictor of 1960 density. Solutions of the 1948 equation were then used as estimates for 1960 at both a ring and district level, resulting in R^2's of 0.784 and 0.743 respectively.

Negative exponential formulations of the general form,

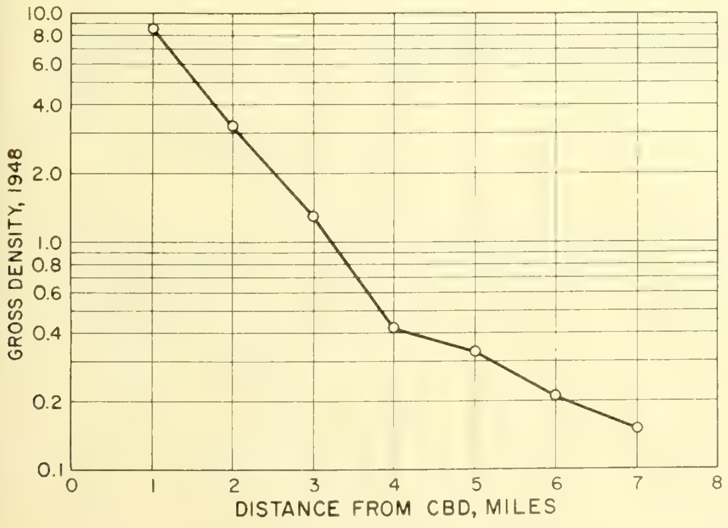

Figure 2.—Semilogarithmic plotting of 1948 gross density by distance bands.

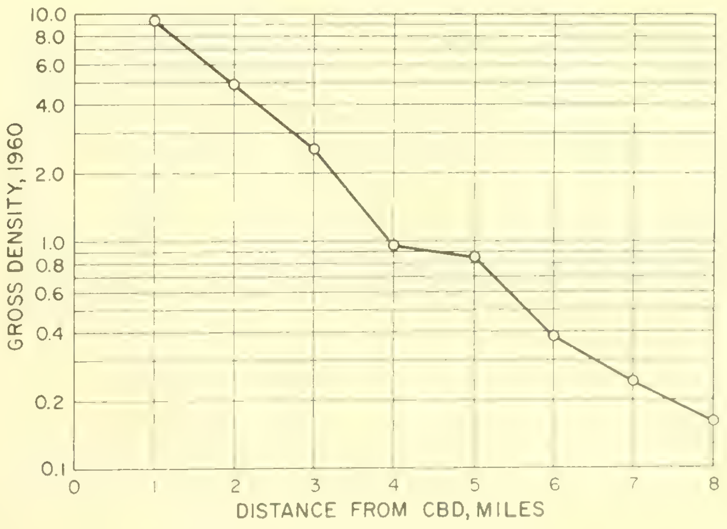

Figure 3.—Semilogarithmic plotting of 1960 gross density by distance bands.

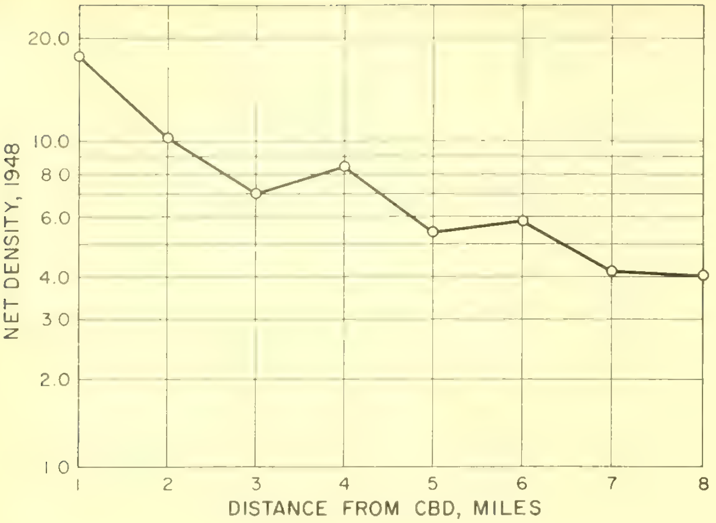

Figure 4.—Semilogarithmic plotting of 1948 net density by distance bands.

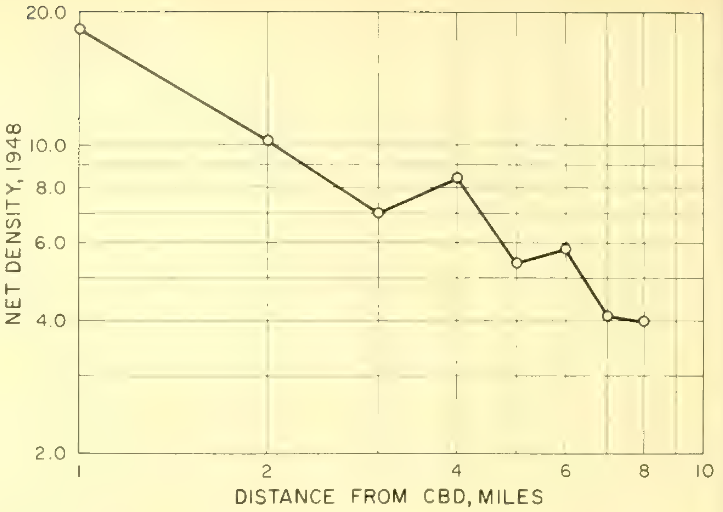

Figure 5.—Logarithmic plotting of 1948 net density by distance bands.

$$D_G = a(X)^b$$

were also investigated as potential gross density gradients, but the calibration and forecast R^2's were consistently below those previously discussed.

Net Residential Density Analysis

Residential land consumption results from an economic equilibrium between supply and demand; selection of a residential location is influenced by the economic condition of the household, its preference pattern for different commodities, the state of the housing market, and the transportation needs of the household (*8, 9, 10, 11*). Household income must be allocated to the purchase of goods and services in an effort to achieve the most satisfaction for the money available. For simplicity assume that household expenditures are divided into three groups: Transportation, housing, and other. If it is assumed that other purchases use a fixed part of the total income, then remaining funds are allocated to housing and transportation in such a way that the household derives maximum satisfaction. To get the best value for his money the head of the household might buy a house where land is least expensive; that is, in an area remote from the urban center. But, although he gets more value for his housing money, he has excessive transportation expenses. The head of another household might locate in an area convenient to transportation, but where housing costs are high thereby forcing him to purchase little housing space. These examples illustrate the causal relation existing between housing costs and transportation accessibility; the most accessible areas are the most desirable and consequently the most costly. The linkage between the decisions of the urban transportation planner and the course of residential development is thus partially illustrated (*12*).

Net density analysis can best reflect the different land-use situations and the relation of land use to transportation; gross density analysis is not so sensitive. The author believes that analytical procedures will be forthcoming which incorporate the relations and provide the transportation planner with practical methods for determining the composition and location of marginal residential development resulting from proposed transportation routes and improvements.

The net density was computed for each district and ring in the study area by dividing the number of dwelling units by the total area of residential land. A more precise measure of average net density can be computed from averaging the density of each individual dwelling unit, but this requires individual household density data. Average net density, as computed for the study presented here, is sensitive to the density variability within each district or ring. Unlike the gross density measure, net density can rise or fall with increases or decreases in the total dwelling units within the total analysis land area.

The plot of the computed ring net densities for 1948 according to the distance of the ring from the CBD generally conformed to the equivalent gross density plot. Figure 4 presents a semilogarithmic plot of these data. However, as suggested partially by the evidence of nonlinearity in the plot in figure 4 and from the study by Kramer (*13*), the net residential density data were plotted logarithmicly as shown in figure 5. Least squares fits were computed for both figures 4 and 5 and R^2's computed. The doubly logarithmic relation proved to be a better linear fit. The linear equation fit to the 1948 net ring densities was:

$$ln\ D_{N.48} = 2.850 - 0.688\ lnX \qquad (7)$$

which transposes to the following form:

$$D_{N.48} = 17.29(X)^{-0.688} \qquad (8)$$

and the R^2 for equation 8 was 0.957. Equation 8 was then examined as a predictor of the 1948 district net densities and yielded an R^2 of 0.835.

Assuming that the net density structure of the study area was stable over the 12-year period, equation 8 was tested as a valid predictor of the 1960 net densities at both the ring and district levels and the resulting values of R^2 were 0.927 at the ring level and 0.844 at the district level. The 1960 net ring densities were then plotted linearly and logarithmically. A linear equation was fit to the logarithmic plot of 1960 net ring densities as shown in figure 6. The resulting equation was:

$$ln\ D_{N.60} = 2.855 - 0.876X \qquad (9)$$

or

$$D_{N.60} = 17.4(X)^{-0.876} \qquad (10)$$

The R^2 for equation 10 was 0.986 and 0.934 at the ring and district levels respectively.

The success in applying equation 8 as predictor of 1960 densities suggested the testing of the following less involved procedure; ring densities in 1960 were estimated to remain exactly as they were computed to be in 1948. This simplifying assumption implies that the added dwelling units over the test period consumed, on the average, the same amount of land as the average dwelling unit existing in the ring in 1948. The computed R^2 for the 1960 net ring densities was 0.849. Carrying this procedure down to the districts, incremental dwelling growth in each district was assumed to locate at the same average 1948 net density as for the particular ring to which it fell. Implicit in this trial is that the intraring net density variability is diminishing over time with each district's net density approaching its ring average. The percent of 1960 net district variance explained utilizing this technique was 0.533. Finally, each district was assumed to maintain constant average net density from 1948 to 1960, the 1948 values then serving as 1960 estimates. An R^2 of 0.640 was computed for this case.

The analysis of the net residential density included the development of multiple regression equations that used selected data as independent explanatory variables. The net density multiple regressions were calibrated at the analysis zone level and were used as predictors for districts and rings. Although strict regression procedure was violated, the resulting errors were estimated to be minor based on results of the interdistrict analysis of variance in relation to intradistrict variance. This procedure required that the dependent variables for each of the regressions be an intensive quantity and thereby independent of the size of the analysis observation unit.

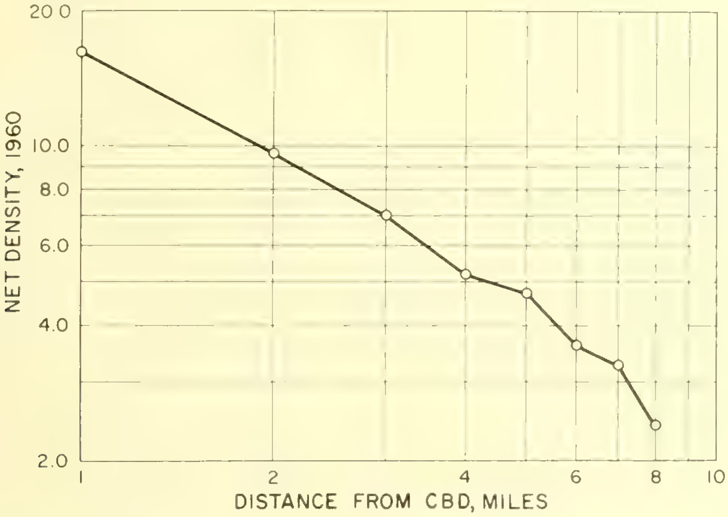

Figure 6.—Logarithmic plotting of 1960 net density by distance bands.

The functional form of the DVRPC density submodel, SPACEC I, was investigated as representative of the density pattern of the region:

$$D_N=a\,e^{\sum_i b_i X_i} \qquad (11)$$

Where,

X_i= Independent variable i.

b_i= Coefficient of variable i.

a= Constant.

D_N= Net residential density.

Equation 11 transforms by logarithmic conversion to the standard linear multiple regression equation:

$$\ln D_N=\ln a+\sum_i b_i X_i \qquad (12)$$

By stepwise regression, least squares equations were developed for 1948 and 1960 net densities. Two final equations were accepted that were logically sound and contained only statistically significant explanatory variables. The 1948 equation computed was:

$$\ln D_{N.48}= 1.534+\underset{(1.92)}{0.005X_1}+\underset{(2.12)}{0.017X_2}+\underset{(6.62)}{0.109X_3} \qquad (13)$$

or,

$$D_{N.48}=(4.64)e^{(0.005X_1+0.017X_2+0.109X_3)}$$

Where,

X_1= Land value per square foot for 1948.

X_2= Percentage of developed land in industrial use for 1948.

X_3= Gross residential density for 1948=$D_{G.48}$

The numbers in parenthesis below each coefficient are the regression t values. Equation 13 was used to estimate the 1948 ring and district net densities by substitution of the appropriate independent variables. The R^2 computed for the rings was 0.802 and for the districts, 0.714.

To test its stability, equation 13 was used as a predictor for 1960, substituting 1960 variables for the 1948 variables. The land value for 1948 (variable X_1) was not replaced because 1960 land value was not available. The R^2's resultant from this prediction were 0.902 and 0.802 for the rings and districts respectively. Solutions to equation 13 were transformed to nonlogarithmic form prior to the calculation of residual errors.

A least squares regression of the general form of equation 12 was then made for the 1960 net ring density distribution. The 1960 least squares equation was:

$$\ln D_{N.60}=0.086+\underset{(6.83)}{0.123X_1}+\underset{(8.21)}{0.542X_2} \qquad (14)$$

or,

$$D_{N.60}=(1.9)e^{(0.123X_1+0.542X_2)}$$

Where,

$D_{N.60}$= Net residential density in 1960.

X_1= Gross residential density for 1960.

X_2= Logarithm of net residential density for 1948=$\ln D_{N.48}$.

The R^2's for the ring and district were 0.805 and 0.832 respectively.

Multiple regression estimates were developed for a nontransformed dependent variable. A stepwise procedure was used to test independent variable combinations and the following equations for 1948 and 1960 were accepted:

$$D_{N.48}=6.25+\underset{(4.04)}{0.085X_1}+\underset{(2.96)}{0.206X_2}+\underset{(6.49)}{0.922X_3} \qquad (15)$$

Where,

X_1= Cost of land per square foot, 1948.

X_2= Percentage of developed land in industrial use, 1948.

X_3= Gross residential density, 1948=$D_{G.48}$.

and,

$$D_{N.60}=1.960+\underset{(2.00)}{0.012X_1}+\underset{(3.12)}{0.053X_2}+\underset{(28.47)}{1.082X_3}+\underset{(7.00)}{0.119X_4} \qquad (16)$$

X_1= Cost of land per square foot, 1948.

X_2= Percentage of developed land in industrial use, 1960.

X_3= Gross residential density, 1960=$D_{G.60}$.

X_4= Net residential density, 1948=$D_{N.48}$.

The R^2's for the ring and district were 0.755 and 0.561, respectively, for equation 15. Equation 15 was then used to predict 1960 ring and district densities. The 1960 independent variables were substituted for 1948 independent variables in equation 15 and R^2's of 0.264 and 0.151 were determined. The R^2's for the ring and district were then computed as 0.963 and 0.938, respectively, for equation 16.

The independent variables and the coefficients in equation 15 seem causally related to the 1948 densities being estimated. The positive coefficient of assessed land cost reflects economic supply and demand; as land cost rises, the individual family must buy less land and, consequently, the net density increases (*8*). The positive coefficient of percentage of industrial land probably reflects the tendency for low income families to settle in marginal residential areas often characterized by industrial development. Preliminary calibration of the SPACEC I model by DVRPC resulted in a positive relation between residential density and industrial activity, (*16*), similar to that found in equations 15 and 16. In contrast, Muth (

states that a negative relation exists between net population density and proximity to local industrial centers; he states that a net decline in housing cost occurs because the neighborhood proximating the industrial area is generally undesirable. However, the findings by Muth suggest two explanations for the positive relation shown in equations 15 and 16. First, the advantage of accessibility to transportation in areas of industrial activity causes an increase in housing cost which is high enough to offset the blighting effect of the industrial environment. Second, areas of industrial activity usually remain in the old sections of the city where housing has a higher density than in new and developing residential areas.

Both explanations are partially supported by data in the analysis zone simple correlation matrix. The variables, accessibility to employment and percentage of land in industrial use, exhibit a moderate positive correlation of 0.37 and substantiate the first explanation given in the previous paragraph. The employment accessibility index is a regional measure in which the built-in colinearity in the two variables is not severe. However, recent findings by Lansing (14) show that the urban worker gives only secondary consideration to employment accessibility when selecting a residence. Therefore as the mobility of the urban worker is increased and the work week is shortened, it is unlikely that the advantage of accessibility to employment would be substantial enough to overcome the nuisance of proximate industrial activity. Further, in small sized urban regions accessibility as a factor in residential location is less influential than in large urban areas.

Although no data were available for the age of industrial development in an area, data for the percentage of land not in use were considered a comparable measure. A negative correlation of 0.38 was found for percentage of land not in use and percentage of land in industrial use. This correlation supports the second explanation for a positive correlation between net residential density and degree of industrial development. The coefficient of gross residential density in equation 15 shows the strong positive correlation between gross density and net density. As more families locate in an area, increased demand for available land will cause increased land cost and density. The gross residential density equation 16 is the same as equation 15, but a fourth independent variable, net 1948 residential density, is used in equation 16. A large amount of the dwelling unit land stock in 1960 existed in 1948, even though development was significant during this 12-year period. The serial correlation between net density variables for 1948 and 1960 is 0.53, which may be a reflection of general inertia in new residential development. That is, the density pattern in an area influences the density of new residential development so that great contrasts in densities within small geographic areas are not common. The correlation of net density variables for 1948 and 1960 is probably dependent on the amount and rate of development. Further investigation of the correlation would require density data for the dwelling units developed during the study period. Such data were not available for the study reported in this article.

Change in Density Structure

Two density distance gradients for equations 3 and 5, given in figure 7, were fit to the ring gross density data for 1948 and 1960. Two changes in the gross density configuration are shown by the deviation of the two linear gradients. First, gross density consistently increased in each ring. The increase was expected as a result of the 52 percent growth in dwelling units in the study area over the

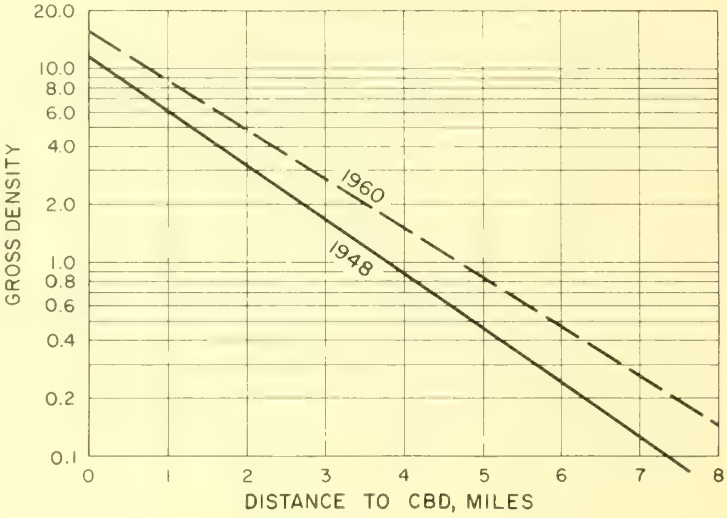

Figure 7.—Calibrated gross density gradients by distance bands for 1948 and 1960.

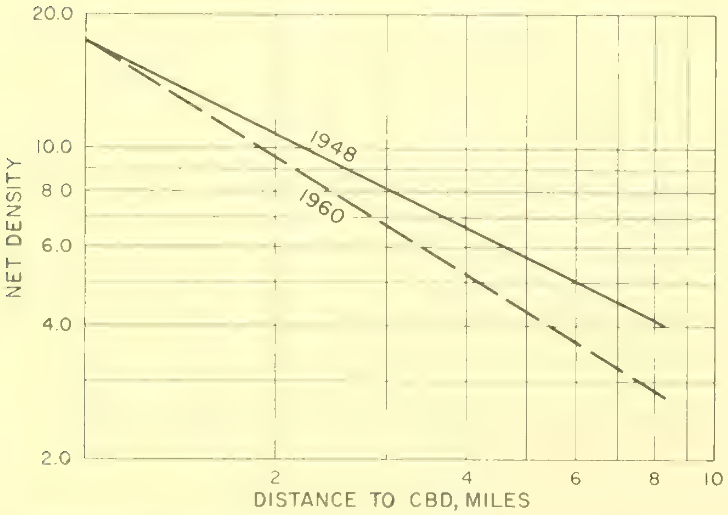

Figure 8.—Calibrated net density gradients by distance bands for 1948 and 1960.

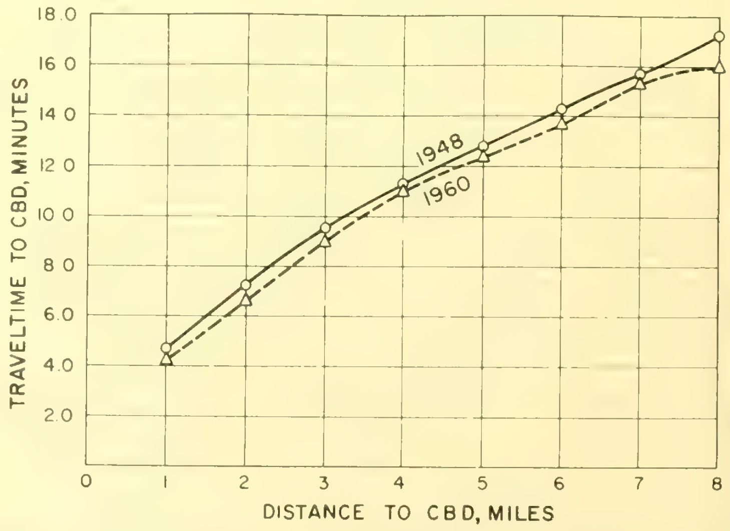

Figure 9.—Traveltimes to the CBD by distance bands for 1948 and 1960.

12-year period. A decline in gross density could only result if the number of dwelling units decreased, and such a decrease is unlikely in a rapidly expanding region. Second, the gross density gradient for 1960 has leveled slightly. The leveling indicates a decrease in population distribution density from 1948 to 1960; the decrease is characteristic of suburban development. Berry, Simmons, and Tennant (15) have observed this leveling situation for Western cities; density gradients tend to decline over time for a given city, and the decline is larger for larger cities.

Net residential density gradients, shown in figure 8, have declined consistently from 1948 to 1960 regardless of distance from the CBD. The decline cannot be explained by absolute population growth; existing market conditions and consumer preferences were influential. Furthermore, a major cause for density decline could be the improvement in transportation service. Figure 9 shows a plot of traveltime to the CBD for each ring for 1948 and 1960. Highway service to the CBD has consistently improved during the 12-year period and, consequently, transportation costs have been reduced. The effects of substantial transportation cost reductions could produce profound shifts in residential patterns owing to possible lower location rents and income effects. Reduced transportation costs would probably result in the use of more living space per household and a decline in net density, as shown in figure 8. However, in spite of continued transportation improvements the decline in net densities is not irreversible; the purchase of additional household space may be halted as the individual households determine that it is no longer to their benefit to acquire more land (*8*). Based on findings by Lansing (*14*) the decline in net residential density can be expected to level. Lansing reports that most households that expressed dissatisfaction with their small lots would prefer to live on lots as large as three-tenths of an acre and that most households that expressed dissatisfaction with lots larger than one-half acre would prefer to live on smaller lots.

Findings

The R^2 entries in table 3 show that the negative exponential formulation, which equates gross density with distance away from the center of the urban area, is an effective measure of the existing gross residential density. Considering that the reported district residual errors are the result of the application of the equation calibrated for the ring, it seems that the intraring variance is relatively minor in comparison with the intraring gross density variability. The negative exponential formulation fitted to the 1948 density distribution was moderately successful as a forecasting device for 1960. The decrease in variance from 1948 to 1960 was 11 percent at the ring and district levels. The distance gradient fit to the 1960 gross density distribution was flatter than that for 1948 and, as shown by the R^2 values, accounted for approximately 11 percent more variance at the ring and district levels. The 1948 gross density distance exponent of 0.648 is probably small for the size of the study area as compared to findings by Muth (*8*).

Results of the net density analysis are contained in table 4. Four procedures were used in the net densities analyses: Distance gradients, multiple regressions with transformed dependent variables, multiple regressions with untransformed dependent variables, and assumed stability of densities. The distance gradients fit to the 1948 net and gross density distributions show significant R^2 at both the ring and district levels. However, accuracy at the district level is substantially less for net density than for gross density analysis. But the stability of the 1948 net density gradient is considerably better than that of the gross density gradient, as shown by the R^2 results when the 1948 density gradient was used to forecast the 1960 density. Explained 1960 ring variance had only a minor decrease and explained district variance increased 10 percent. The increase in district variance undoubtedly reflects the peculiarities possible when linear estimations are forced through nonlinear data by the expedient of logarithmic transformation. The R^2's for the 1960 density gradient were high and almost exactly matched the equivalent gross density gradient. The slope of the net density gradient increased from 1948 to 1960. Overall, the density distance gradient formulations provided comparable accuracy at the district and ring levels for net and gross residential densities, and the 1948 net density gradient was shown to be an accurate predictor of 1960 conditions.

The multiple regression R^2's for the net density distribution are also presented in table 4. Moderate success was achieved in calibration for the 1948 nonlinear formulation of net density. The residual variance was larger for the net density distributions than for the distance gradient at both the ring and district levels, and the explained variance for the ring analysis decreased. But, unexpectedly, the analysis resulted in higher R^2 for forecasts than it did in calibration. Solutions to the 1948 equation with the 1960 independent variables, with the exception of land cost, had less residual error after transformation to linear form than the equation calibrated to the 1960 data. The results in table 4 for the linear regression estimating equation show that the accuracy of the 1948 calibration and forecast is inferior at the district and ring levels to the nonlinear regression and distance gradient formulations although the same independent variables are used in the linear and nonlinear 1948 regressions. However, the calibration R^2's for the 1960 linear regressions are comparable to those for the distance gradient and better than those for the nonlinear regression formulation. The strong serial correlation between 1948 and 1960 net densities probably accounts for the sharp increase in accuracy of the 1960 equation as compared to that for 1948 inasmuch as both equations use the same independent variables, except that the 1948 net density is included in the 1960 equation.

The stability of the net density pattern over the 12-year period explains to a large extent the results obtained when the 1948

Table 3.—Calibration and forecast R^2's for gross density distance gradients

Equation	R^2, 1948		R^2, 1960	
	Ring	District	Ring	District
$D_{G\cdot48}$ $11.36e^{-0.648X}$	0.886	0.834	[1] 0.784	[1] 0.743
$D_{G\cdot60}$ $15.58e^{-0.58X}$	....	....	0.989	0.923

Forecast results.

Table 4.—Calibration and forecast R^2's for net density analysis

Analysis procedure	Model form	1948		1960	
		Ring	District	Ring	District
Distance gradients	$D_{N\cdot48} = (17.3)X^{-.688}$	0.957	0.761	[1] 0.927	[1] 0.844
	$D_{N\cdot60} = (17.4)X^{-.875}$	-		0.986	0.934
Log linear multiple regression.	$\ln D_{N\cdot48} = 1.534 + 0.005$ Land value $+ 0.017\%$ Industrial land $+ 0.109$ Gross density$_{48}$	0.802	0.714	[1] 0.902	[1] 0.820
	$\ln D_{N\cdot60} = 0.086 + 0.123$ Gross density$_{48}$ $+ 0.542 \ln D_{N\cdot48}$			0.805	0.832
Linear multiple regression.	$D_{N\cdot48} = 6.257 + 0.085$ Land value $+ 0.206\%$ Industrial land $+ 0.922$ Gross density$_{48}$	0.755	0.561	[1] 0.264	[1] 0.151
	$D_{N\cdot60} = 1.960 + 0.012$ Land value $+ 0.053\%$ Industrial land $+ 1.082$ Gross density$_{60}$ $+ 0.119\ D_{N\cdot48}$	-	-	0.963	0.938
Assumed no change in net densities.	Assume 1960 ring densities same as 1948 ring densities.	.....	-	[1] 0.849	[1] 0.533
	Assume 1960 district densities same as 1948 district densities.	-	-	-	[1] 0.640

[1] Forecast results.

ring and district densities were assumed to hold for 1960. Considerable net density variability can be explained as a carryover from the base time period. However, only 64 percent of the variance is accounted for when 1960 district densities are assumed to be the same as 1948. The assumption of stability in the density gradient equation was a more accurate forecasting procedure than that of projecting no change in the density distribution. However, by simulating the effects of temporal changes in the urban region, through the use of properly structured regression equations, density variability can be expected to be more accurately explained.

The results given in tables 3 and 4 indicate that the analysis and projection of net residential densities can be made as accurately as for gross density. Success can be expected in forecasting urban net densities by simple distance gradients. Results obtained from the multiple regression equations were somewhat inconsistent; the nonlinear regression equation was superior to the linear equation in calibration to 1948 conditions and in forecasting to 1960, but the linear equation was superior in calibration for 1960. Significant regression equations containing rational explanatory variables can be developed. The addition of a household income measure as a variable in regression equations might contribute to the investigation of urban residential density. However, the income data available for the study reported in this article were census tract medians that were too aggregated to be consistent with the analysis zone level of the other data used in the study.

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(*13*) *Population Density Patterns*, by Carol Kramer, C.A.T.S. Research News, vol. 2, No. 2, Jan. 31, 1958, pp. 2–10.

(*14*) *Residential Location and Urban Mobility: The Second Wave of Interviews*, by John B. Lansing, Survey Research Center, Institute for Social Research, University of Michigan, January 1966.

(*15*) *Urban Population Densities: Structure and Change*, by Brian J. L. Berry, James W. Simmons, and Robert J. Tennant, Geographical Review, July 1963, pp. 389–405.

(*16*) *SPACEC I Parameters*, internal memorandum Delaware Valley Regional Planning Commission, Feb. 2, 1966.

New Publications

(*Continued from p. 241*)

The volume contains information on two types of reinforced concrete retaining walls subject to various loading conditions. The first type is cantilever retaining walls on spread footings designed for four different loading conditions. The walls range from 6 feet to 30 feet in height. Pile foundation alternates were prepared for the first two cases of loading and are suggested for use where materials of adequate strength are too deep for economic use of spread footing design, but where the upper strata around the piles have sufficient strength to provide the required lateral support for the piles. The second type is counterforted retaining walls founded on spread footings designed for two different loading conditions. The walls range from 18 feet to 36 feet in height.

Use of Riprap for Bank Protection

Use of Riprap for Bank Protection, published in June 1967 (40 cents a copy), is No. 11 in the Bureau of Public Roads series of hydraulic engineering circulars. It discusses the design and construction of riprap for bank protection and contains sample specifications for different types of riprap. Information on the design and construction of filter blankets is included.

The principal source of material for this circular is the summary of slope protection methods in Proceedings of the American Society of Civil Engineers, vol. 74, 1948. This source is complemented by more recent research on protecting earth dams particularly the effect of gradation of stone on riprap performance.

Also available from the U.S. Government Printing Office are Hydraulic Engineering Circular No. 5 (1965), *Hydraulic Charts for the Selection of Highway Culverts* (45 cents), and Hydraulic Engineering Circular No. 10 (1965), *Capacity Charts for the Hydraulic Design of Highway Culverts* (65 cents).

ERRATA

In the second installment of the article, *Capacity Analysis Techniques for Design of Signalized Intersections*, which appeared in the October issue of PUBLIC ROADS, *A Journal of Highway Research*, vol. 34, No. 10, three of the illustrations contained errors as noted below.

Figure 17:

- In APPROACH N table, for movement NW under heading CHART REFERENCE—*18–B, –C* should read *17–D, Fig. 9*.
- In APPROACH N table, for movement NE under heading G/C REQ'D—*0.21* should read *0. 17*
- In APPROACH W table, for movement WN under heading CHART REFERENCE—*18–B, –C* should read *17–B, –E*.
- In APPROACH E table, under heading REMARKS—$D_a=225$ should read $D_a=275$.

Figure 19:

- On Ramp D—*520* should read *540*.

Figure 22:

- Table titles *DE–FG* should read *EF–GH* (2 places).
- In table INTERSECTION *EF–GH*, AM PEAK, under heading APPROACH OR MOVEMENT—*EE* should read *EF*.

Figure 26:

- In table INTERSECTION *EF–GH*, PM PEAK, under heading POSSIBLE CAPACITY—*870* should read *1400*.

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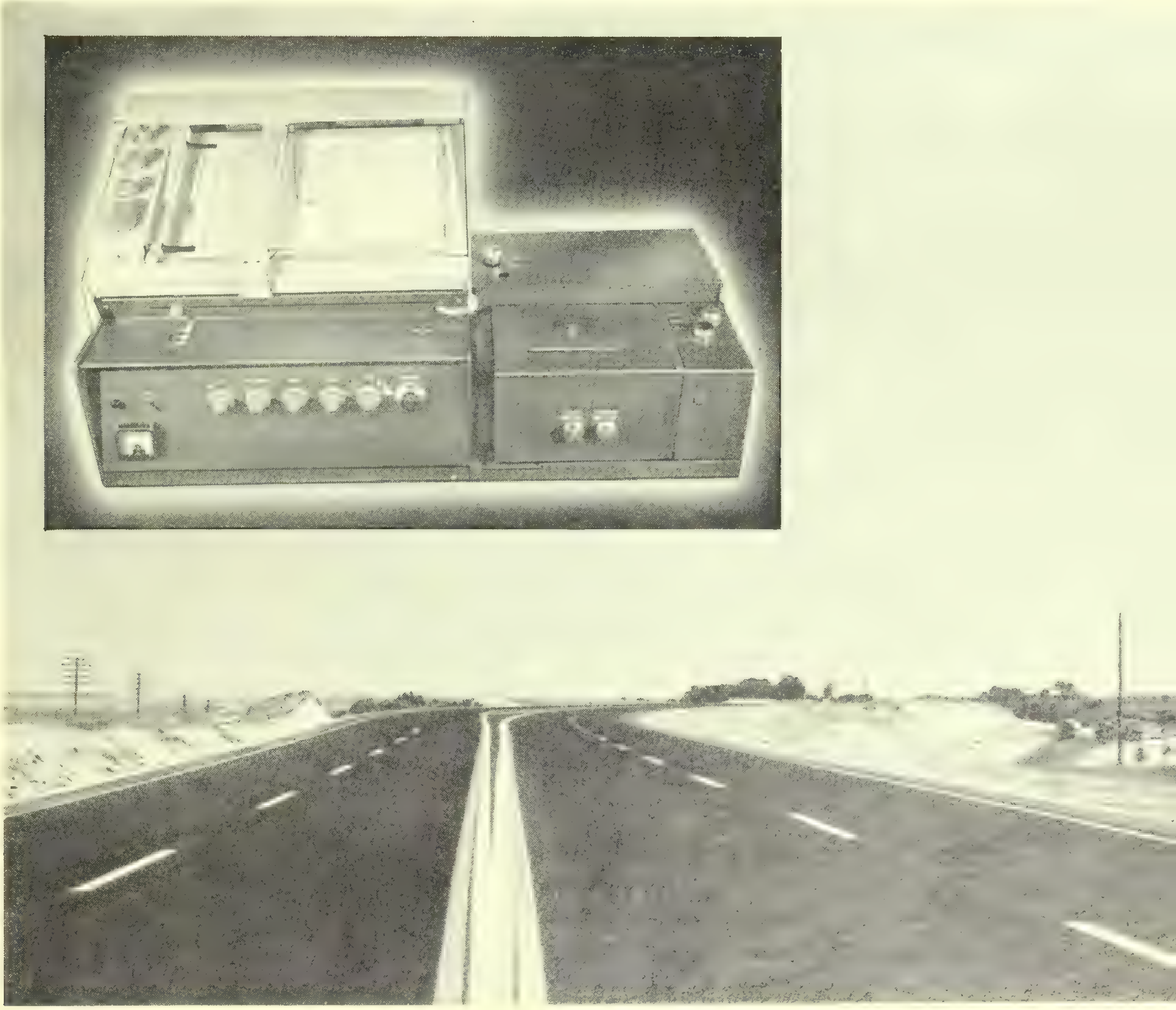

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Ultraviolet Spectrophotometric Detection of Adulteration in Traffic Paint Vehicles

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by[1] LEONARD BEAN, Research Chemist, and BERNARD CHAIKEN, Principal Research Chemist, Materials Division

Based on the materials investigated in a research study by the Bureau of Public Roads, an ultraviolet spectroscopic method for detecting adulteration of traffic paint vehicles in amounts as small as 2 to 5 percent by weight of the vehicle solids is reported herein. The method is applicable to the monitoring of purchased lots of traffic paint after a reference sample has been road tested and accepted for use. In the detection method used, the paint pigment is removed by centrifuging. A separate determination of nonvolatile solids is made by heating at 105° C. The solvents from a separate test sample are removed from a weighed sample of the vehicle by means of a rotating flask vacuum evaporator. A spectral grade of cyclohexane is used as a solvent to quantitatively prepare solutions of the vacuum dried vehicle solids, and the absorbances of the resultant solutions are measured in a double-beam recording ultraviolet spectrophotometer. The spectral curves and characteristics of the vehicle solids from both the reference and the purchased paints are compared to establish uniformity or evidence of adulteration. In the study reported here an alkyd resin was used as it is the most representative of traffic paint vehicles in use today. Plots of percentage of each adulterant—wood rosin, hydrocarbon resin, fish oil, and tall oil—against spectral absorbance at 264 nanometers (nm.) (or millimicrons) show the effect of the adulterant. About 4 hours are required for a single determination. Modifications of the method developed appear to have potential applicability to protective coatings in general, as well as to traffic paints, where composition is specified.

Introduction

APPROXIMATELY HALF of the State highway departments purchase traffic paints on the basis of extended road performance tests. When the best performing and most economical paint in terms of cost per useful day of service has been selected, it is essential that the paint delivered for actual use have the same composition as the test sample used in the road performance test. Many instances have been reported in which the delivered paint failed to have satisfactory durability or drying time. Adulteration of the delivered paint vehicle was found to be the cause. Consequently, State highway officials and reputable paint suppliers have been increasingly concerned about the problem of traffic paint adulteration. Unfortunately, detection of certain types of adulteration is difficult, tedious, and sometimes impossible when the classical chemical means normally available are used. Therefore, interest has increased in the development and use of rapid instrumental methods to detect adulteration of paint vehicles.

Adulteration of the paint pigment now can be quickly assessed by means of X-ray diffraction (*1*)[2] and the paint volatiles can be rather easily analyzed by infrared spectrometry. However, the detection of adulteration of the third major component of the paint, vehicle solids, is more difficult and is the basis for the study reported herein.

Applicability of Infrared Analysis and Other Methods

Infrared spectral analysis is a rapid and reliable way of detecting gross adulteration of the paint vehicle solids or the substitution of a completely different generic type of vehicle solid for the one in the test or reference sample. Representative infrared spectra of various generic types of vehicle constituents are available in the literature (*2*, *3*, *4*). These

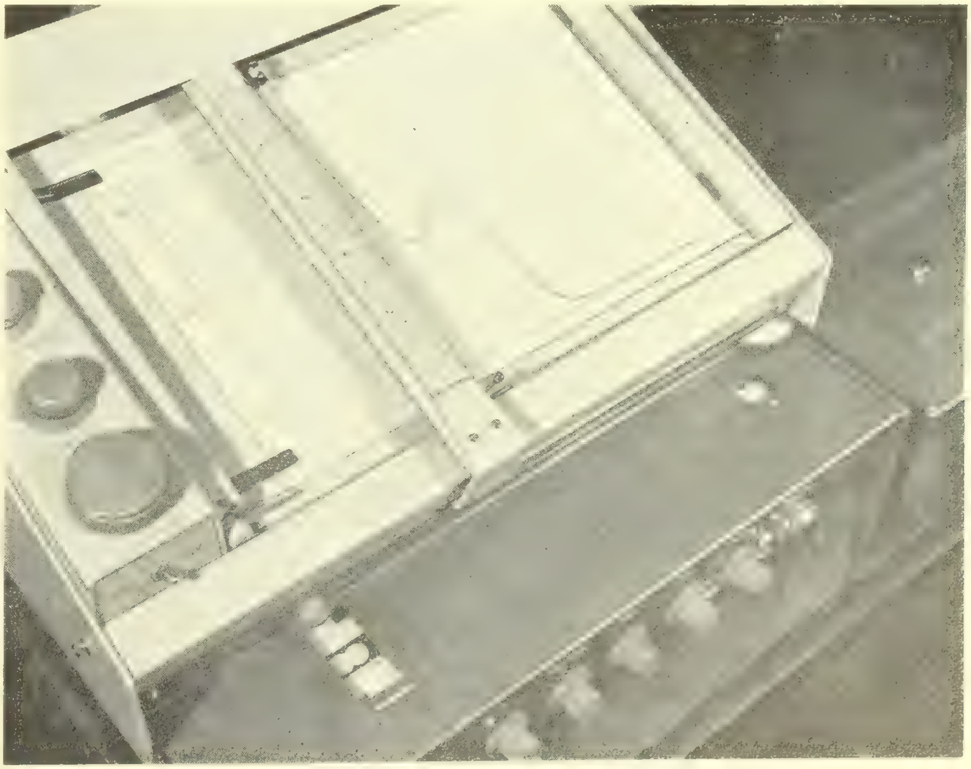

Figure 1.—Ultraviolet spectrophotometer used to measure ultraviolet absorbance.

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[1] This article contains essentially the same material that was reported in *Materials, Research and Standards*, February 1967, published by the American Society for Testing and Materials.

[2] The italic numbers in parentheses indicate the references listed on p. 266.

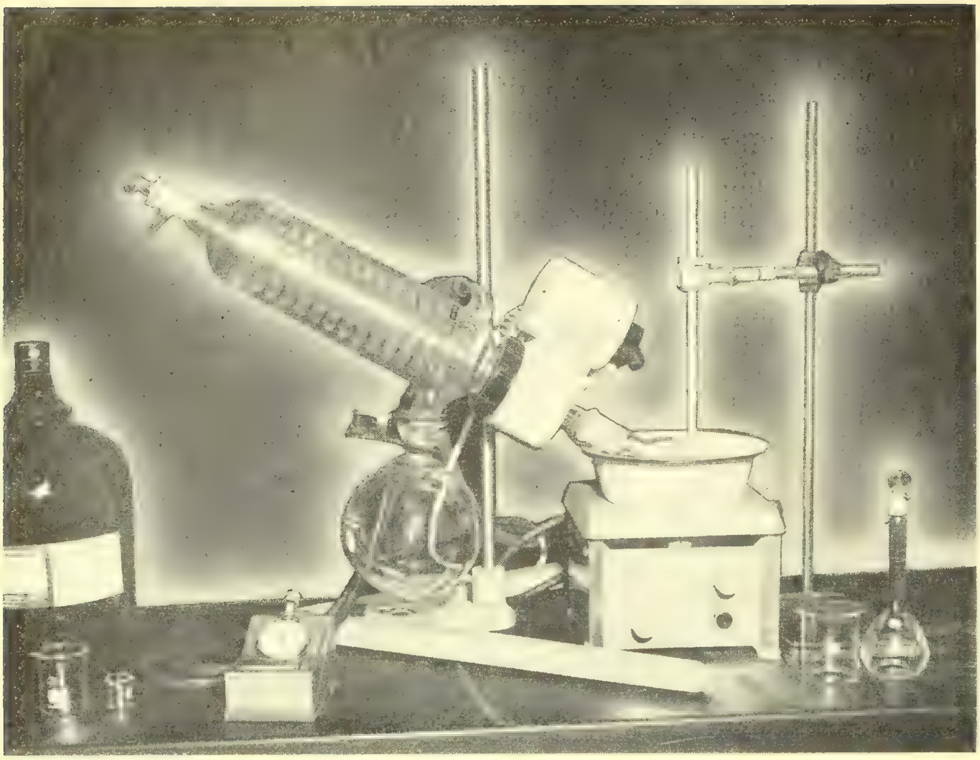

Figure 2.—Rotary-flask vacuum evaporator for removing solvents from paint vehicle solids.

spectra are usually adequate for identifying such adulteration or substitution. However, minor adulterations of some vehicle solids with less expensive raw materials such as fish oil, rosin, tall oil, and hydrocarbon resin are often difficult to detect by such means. These may and do have legitimate uses in some types of coatings. However, for the purpose of this study, they were considered adulterants in order to determine to what degree alien components could be detected in an alkyd paint vehicle. Recent cooperative investigations as yet unpublished by Subcommittee 44 on Traffic Paints of Committee D–1 of the American Society for Testing and Materials, have shown that additions of such extraneous substances to oil-alkyd paints, even up to the level of 20 percent by weight of the vehicle solids, cannot be clearly detected by infrared spectral analysis. Gas chromatography can be useful in detecting adulteration of the drying-oil portion of vehicle solids. However, the lengthy procedures required for gas chromatography and its limited application only to the drying-oil part of the vehicle solids indicated a need for other procedures for detecting adulteration in paint vehicles. As ultraviolet (UV) spectral analysis is known to be an extremely sensitive quantitative tool that rapidly reveals minor differences between organic substances, its potential was explored for detecting adulteration of vehicle solids commonly used in traffic paints.

In this study a comparison was made between UV spectral measurements of specially prepared solutions of alkyd resin containing no adulterants and spectral measurements of similar solutions containing known amounts of adulterants. A soya-alkyd resin was selected because it is a type of paint vehicle most frequently used by many State highway departments. Spectral absorbance curves of these solutions were obtained by means of a recording ultraviolet spectrophotometer.

Equipment and Materials

A recording double-beam ultraviolet spectr photometer (see fig. 1) equipped with centimeter matched silica cells was used t obtain spectral curves and measure the U absorbance. The light source was a hydroge discharge lamp and the detector was a phot multiplier tube. A rotary-flask vacuum eva orator of the type shown in figure 2 was use for removing the solvents from the vehicl solids.

Materials used in the study are identifie as follows:

- *Alkyd resin solution.*—A 1-gallon sample of medium oil length soya-alkyd resin havin the following specification description: Aci value, 4–10 (solid basis); color (Gardner), maximum; Stoke viscosity, 5.5 to 8.8 viscosity (Gardner-Holdt), T to V; solids 49 to 51 percent; average weight per gallon 7.60 pounds.
- *Wood rosin.*
- *Hydrocarbon resin.*
- *Fish Oil.*—A sample of liquid materia labeled, *Product, Light Cold Pressed Fis* having the following specification description: Acid value, 8 maximum; color (Gardner), 12 maximum; cold test, 2 hours a 32° F., clear; average weight per gallon 7.69 pounds.
- *Tall oil.*
- *Spectral grade cyclohexane.*

Experimental Procedure

In some preliminary experiments the pigment from an oil alkyd paint was removed by centrifuging. Total nonvolatile solids were determined by heating for several hours a

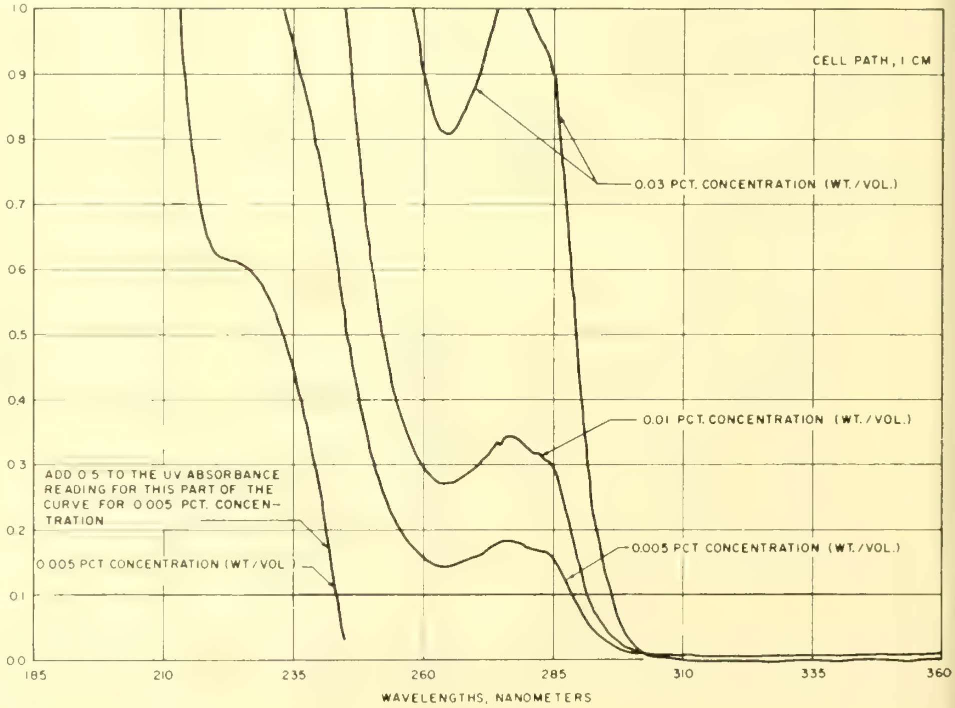

Figure 3.—Ultraviolet absorbance curves for different concentrations of alkyd resin solutions in cyclohexane.

05° C. The residue after heating under these conditions was not suitable for ultraviolet absorbance measurements. As would be expected, the heat treatment and exposure to air had polymerized the material so that its re-solution was difficult in any usable spectral grade solvent. Also, the polymerization caused the UV absorption characteristics of the material to change considerably, and the amount of change varied in relation to the extent of heating. Some attempts were made to remove the volatile solvents from the vehicle by heating it at 60° C. in a bell jar under an inert atmosphere of carbon dioxide. The resultant spectral pattern varied with time of heating and was not repeatable to a satisfactory extent. Therefore, the use of a rotary-flask vacuum evaporator was investigated. Evaporation under vacuum of approximately 560 mm. of mercury for 20 minutes with the rotating flask partially immersed in a water bath maintained at a temperature of 56° to 60° C. gave resultant vehicle solids that were soluble in cyclohexane and had repeatable ultraviolet absorption characteristics suitable for quantitative purposes. Absorbance measurements on a solution of alkyd resin in cyclohexane made immediately after its preparation and after the solution had stood for 1 week showed no evidence of appreciable change or precipitation of the resin from solution.

Several different spectral grade solvents were evaluated. Some compromise had to be made between the dissolving power of the solvent for the dried test resin, and the solvent's UV absorbance characteristics from 210 to 360 nm., the range studied. Spectroanalyzed cyclohexane was selected for the work reported here. Although 20 percent of the wood rosin remained insoluble in cyclohexane, this solvent completely dissolved the other materials studied. Methanol dissolved all the wood rosin but it did not dissolve the alkyd resin and, therefore, was not applicable. Absorbance characteristics of the cyclohexane from 210 to 360 nm. were excellent. Other solvents tried were *p*-dioxane, *n*-butyl alcohol, isobutyl alcohol, *n*-heptane, acetonitrile, *n*-hexane, isopropanol, glycerine, and tetrahydrofuran. Only tetrahydrofuran completely dissolved all the materials studied. However, tetrahydrofuran usually contains hydroquinone or other substances that have been added to inhibit its oxidation. UV measurements made with this solvent indicated that either the tetrahydrofuran or the inhibitors were reacting with the dissolved alkyd resin, thereby affecting the spectral characteristics. When such solutions were allowed to stand, substantial changes occurred in UV absorbance.

After these preliminary experiments, a study was made of the repeatability of the results from the procedure tentatively selected. For these tests a portion of the alkyd resin solution was weighed, the volatile solvents in the sample were removed by evaporation under inert conditions in a rotating-flask vacuum evaporator, and the alkyd resin was redissolved in spectral grade cyclohexane. This solution was transferred to a volumetric flask; appropriate aliquots of this solution were taken to prepare solutions containing 0.03, 0.01, 0.005, and 0.001 percent concentrations of dry alkyd resin. For the purpose of this report, the percent concentration means the weight of solute per 100 ml. of solution; for example, a 0.03 percent solution means 0.03 grams of solids in 100 ml. solution. The UV absorbances for the various solutions containing different concentrations of alkyd resins were measured. Typical absorption patterns are shown in figure 3. The peak at 276 nm. is reported to be caused by phthalic acid (*5*). Within a 12-day period, eight replicate runs were made. The absorbances obtained at 276 nm. for 0.03, 0.01, and 0.005 percent solution concentrations of the alkyd resin in cyclohexane are given in table 1. Replicate ab-

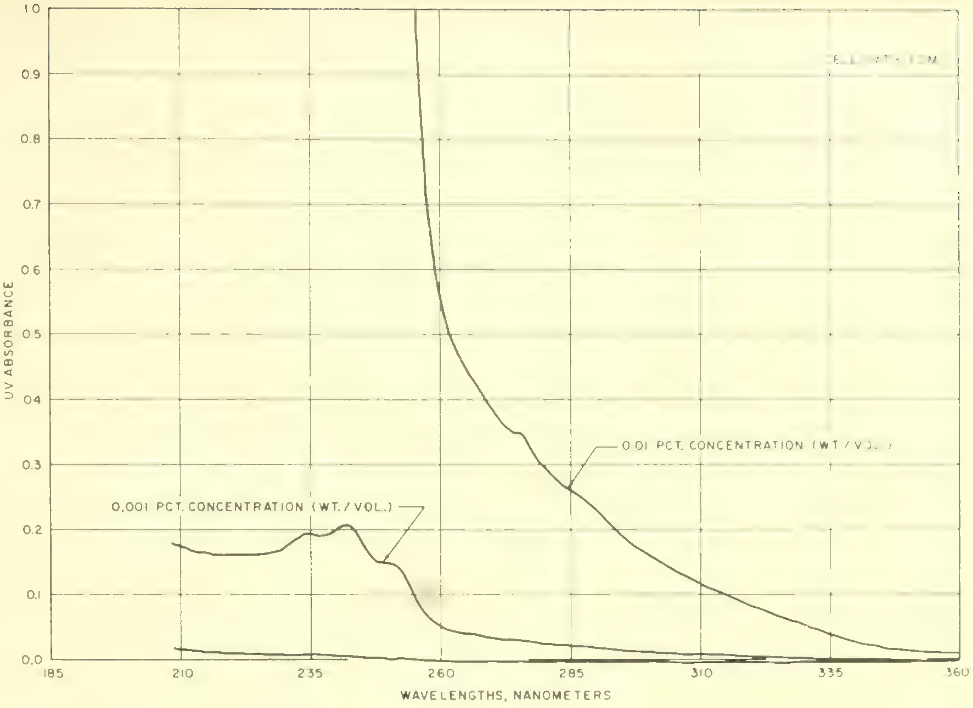

Figure 4.—Ultraviolet absorbance curves for different concentrations of wood rosin in cyclohexane.

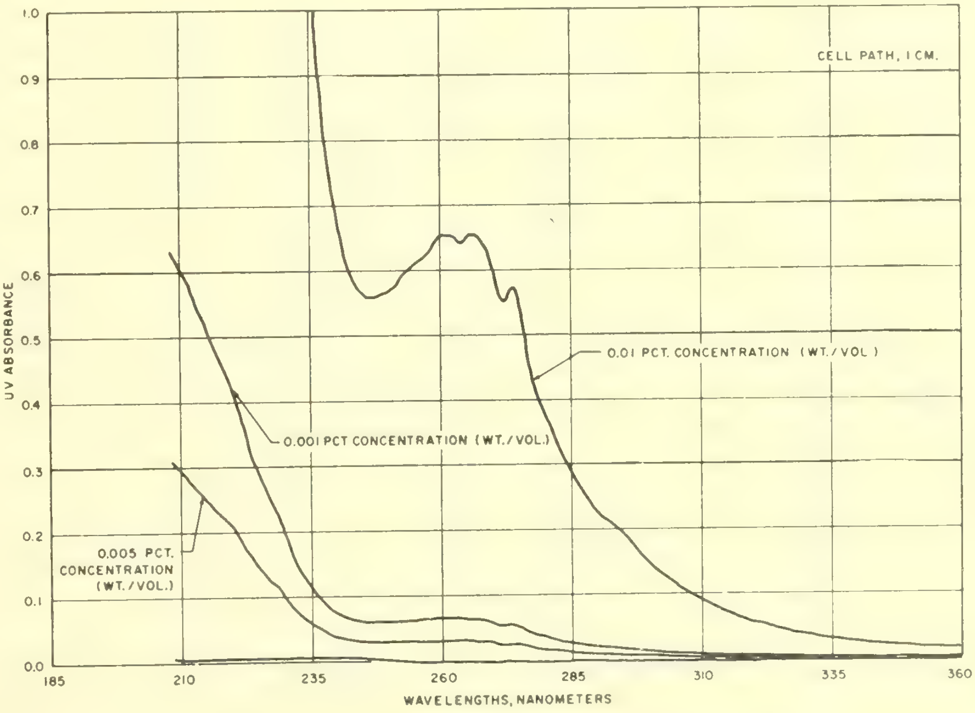

Figure 5.—Ultraviolet absorbance curves for different concentrations of hydrocarbon resin in cyclohexane.

sorbances obtained at 264 and 225 nm. are also given. In the spectrum of alkyd resin, a spectral trough occurs at 264 nm. and a shoulder occurs at 225 nm.

The calculated average, range, standard deviation, and coefficient of variation in the absorbance values for each of the wavelengths and concentrations studied are also given in table 1. Based on the results at 276 nm., the relative spectral repeatability was somewhat better for solutions containing the greater amount of resin.

After the absorbance patterns of the unadulterated alkyd resin solutions had been established, absorbance patterns were made of cyclohexane solutions of the adulterants to be used: wood rosin, hydrocarbon resin, fish oil, and tall oil. These patterns are shown in figures 4, 5, 6, and 7. It must be remembered that part of the wood rosin would not dissolve in cyclohexane. Consequently, solutions of wood rosin in cyclohexane were prepared and filtered through a bed of glass wool in order to remove the insoluble fraction before the stock solution was prepared, and the procedure of taking aliquot parts of the solution and measuring absorbances was continued. Glass wool was used as a filter because cyclohexane washed through ashless filter paper gave a definite ultraviolet absorption pattern.

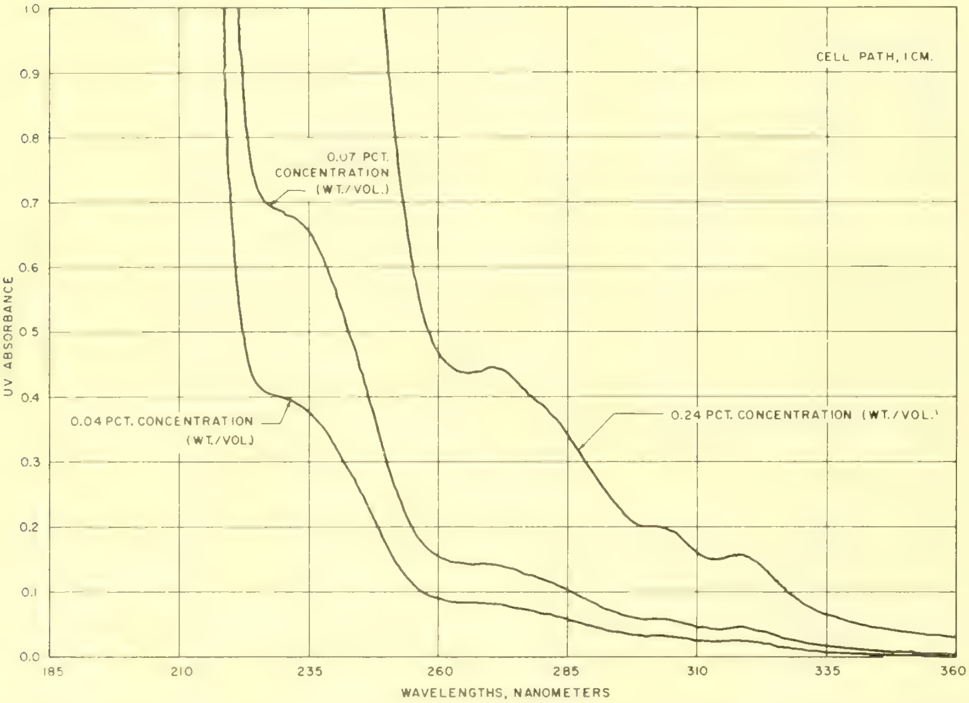

Figure 6.—Ultraviolet absorbance curves for different concentrations of fish oil in cyclohexane.

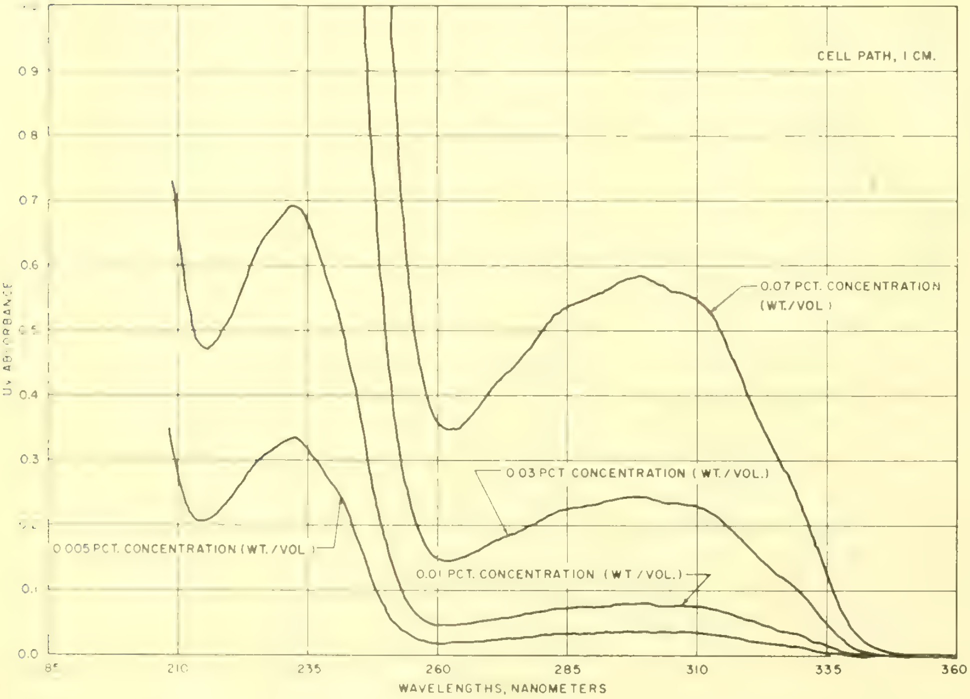

Figure 7.—Ultraviolet absorbance curves for different concentrations of tall oil in cyclohexane.

By using a light box and superimposing the recorded spectral pattern of each adulterant over the pattern of the alkyd resin, and keeping in mind any differences in concentrations, a good idea of the effect of each of the adulterants on the resulting pattern of an alkyd resin-adulterant mixture could be obtained. Always, the overall spectral characteristics of each of the adulterants differed substantially from the alkyd resin and, therefore, an ultraviolet spectral approach to detecting alkyd resin adulteration seemed assured. The questions were then: How small an amount of adulterant could be detected, and how could quantitative measurements be made? A decision was made to focus upon the extent to which the alkyd resin absorbance peak at 276 nm., or its absorbance trough at 264 nm., would be changed by added increments of adulterant. These wavelengths were selected because sharp spectral peaks or troughs lend themselves more readily to quantitative treatment.

Effect of adulterants

From the comparison of the spectral patterns, it appeared that the effect of each of the adulterants studied would be to alter the alkyd resin absorbance at its trough at 264 nm., but the absorbance at its peak at 276 nm. would not always be altered. For example, the absorbance of wood rosin coincided almost exactly with the absorbance of the alkyd resin at its 276-nm. peak when the same concentrations were used. Referring to wood rosin in figure 4, the peak at 241 nm. is attributed to abietic acid, the shoulder at 250 nm. is attributed to isoabietic acid, and the shoulder at 276 nm. is attributed to dehydroabietic acid (*6*). The shoulder at 276 nm. coincides with the reported peak of phthalic acid in the alkyd resin (*5*). Therefore the alkyd resin absorbance *trough* at 264 nm., rather than the adjacent peak was used in order to preclude the possibility of permitting wood rosin adulteration to go undetected.

Sample preparation

Solutions of adulterated alkyd resin paint vehicles were prepared so that the vehicle solids part contained 5, 10, and 20 percent each of wood rosin, hydrocarbon resin, fish oil, and tall oil. Total nonvolatile solids content of each adulterated solution was determined by heating samples at 105° C.

Separate samples of the adulterated solutions were weighed and heated in the rotating flask evaporator under line vacuum to remove the solvent in the test sample. The resultant solids were redissolved in cyclohexane, and solutions containing 0.03, 0.01, 0.005, and

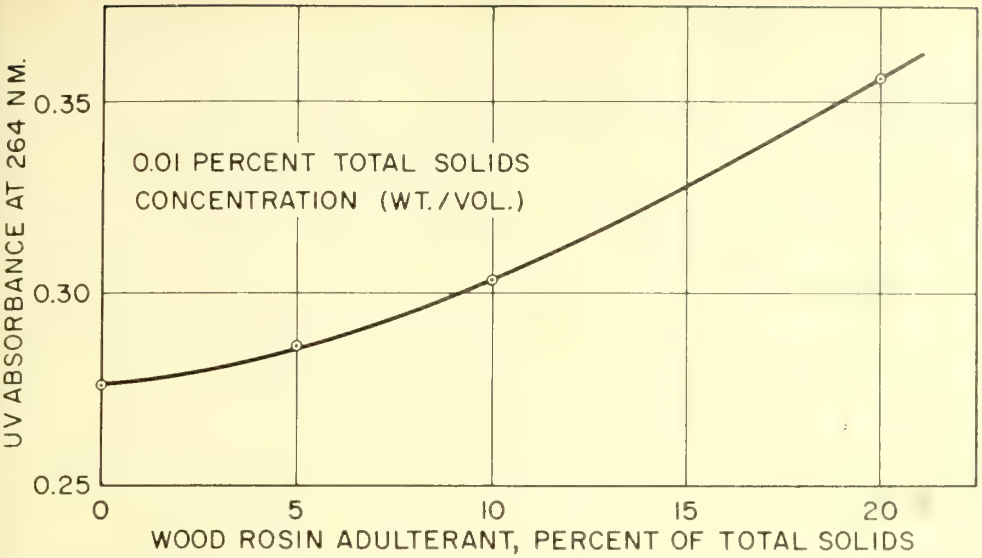

Figure 8.—Effect of wood rosin adulteration of alkyd resin on its UV absorbance.

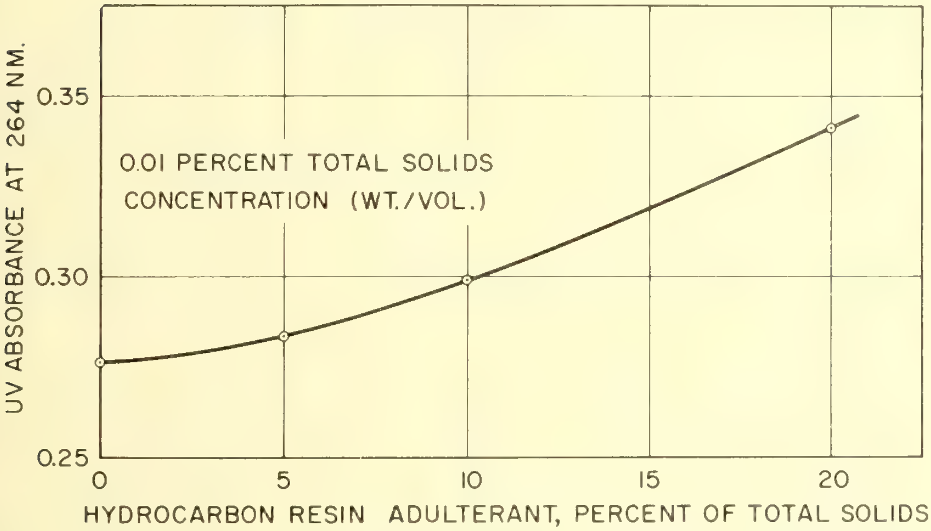

Figure 9.—Effect of hydrocarbon resin adulteration of alkyd resin on its UV absorbance.

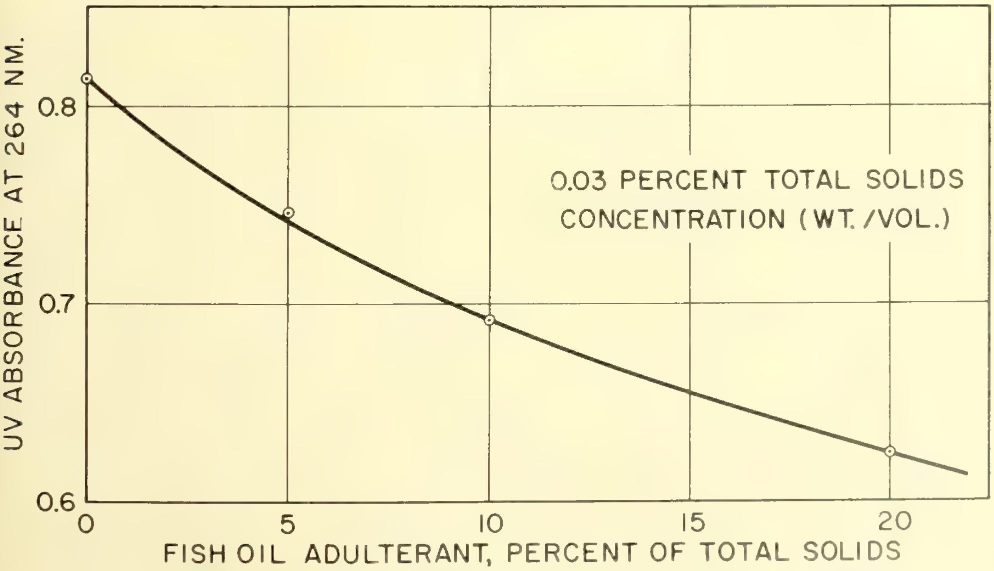

Figure 10.—Effect of fish oil adulteration of alkyd resin on its UV absorbance.

0.001 percent concentrations of nonvolatile solids were prepared by taking aliquot parts. UV absorbance measurements for each of the solutions were then made.

The effects of each added adulterant on the absorbance of the alkyd resin at the 264-nm. trough are shown in figures 8, 9, 10, and 11. Each of the points shown was obtained from a single determination. The superimposed patterns of 0.03 percent concentrations of the alkyd resin mixed to contain 0, 5, 10, and 20 percent fish oil respectively, on the basis of vehicle solids, are shown in figure 12 for the spectral area near 264 nm.

Four runs were made on the alkyd resin sample that had been mixed with 10 percent tall oil to check further the repeatability of the method. For a total solids concentration of 0.03 percent, a coefficient of variation of 1.35 percent was obtained. This is within the repeatability previously obtained from eight runs on the unadulterated alkyd resin at the same total concentration and wavelength.

From consideration of the results obtained on the adulterated samples with the calculated variability for unadulterated alkyd resin as given in table 1, minor adulteration of alkyd resins can be expected to be detected by ultraviolet spectroscopy. Allowing for a 2-sigma deviation within any single set of determinations at a single wavelength of 264 nm. (for the reference paint vehicle and test paint vehicle) the following levels of adulteration should be detectable at a 95 percent confidence level: tall oil, 2 percent; fish oil, 2 percent; wood rosin, 5 percent; and hydrocarbon resin, 5 percent. These detection limits, especially for wood rosin and hydrocarbon resin, can be reduced considerably by spectral changes at other wavelengths. For example, for wood rosin and hydrocarbon resin, large spectral differences exist between each of these materials and the pure alkyd resin in the area of 210 to 235 nm. (figs. 3, 4, and 5). By judicious use of these spectral differences and other parts of the spectrum, the detection limits for adulteration can be reduced considerably.

Figures 8 through 11 show that adulteration with wood rosin or hydrocarbon resin increases the absorbance of the alkyd resin trough at 264 nm., and the presence of fish oil or tall oil reduces the absorbance value at that same wavelength. Presumably, a combination of two adulterants could be calculated and added to alkyd resin so that UV absorbance measurements at only one wavelength might not be sufficient to show that adulteration had occurred, if conclusions were based on this characteristic only. However, as previously stated, a comparison of the entire curves of reference and test materials provides qualitative information that helps to identify such a combination of adulterants. For example, an appreciable spectral difference was noted in the curves for 0.005 percent solutions of each of the following: pure alkyd resin, alkyd resin mixed with 10 percent tall oil, and alkyd resin mixed with 10 percent wood rosin.

The unadulterated alkyd resin curve reached an absorbance of 1 at about 233 nm. while the curve for 90 percent alkyd resin and 10 percent

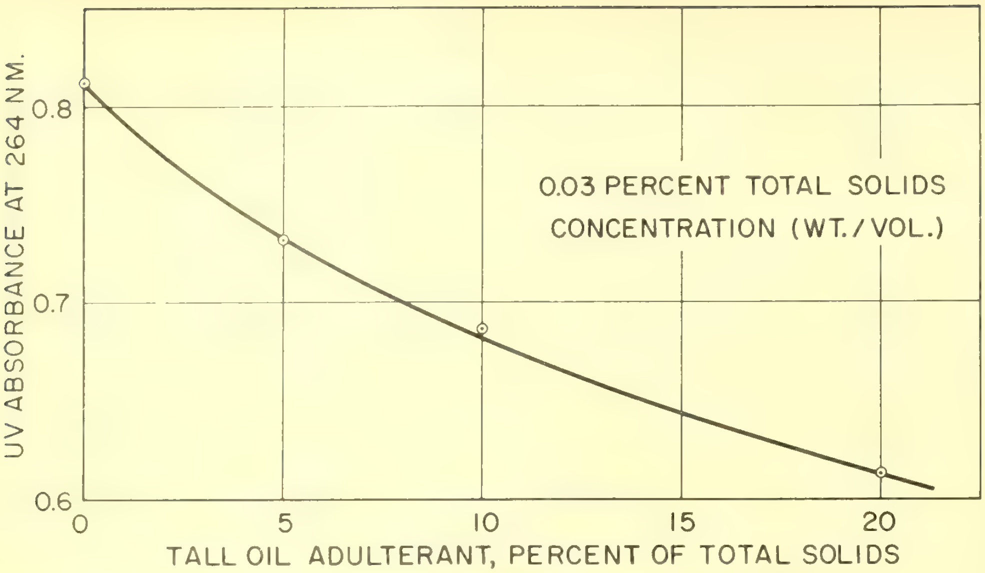

Figure 11.—Effect of tall oil adulteration of alkyd resin on its UV absorbance.

wood rosin reached an absorbance of 1 at 231 nm. In contrast, alkyd resin with 10 percent tall oil, reached this absorbance between 216 and 223 nm. in the case of the four runs made in this adulteration. Presumably, sophisticated adulteration with a combination of tall oil and wood rosin could be detected from a significant wavelength shift of the spectral curve where it reaches the higher absorbance values.

Because of the time required to conduct the study, the stability of the test resin on storage was of interest. Tests showed that a small change in absorbance characteristics of the unadulterated alkyd resin had occurred after a 3-month period. A slightly greater change occurred in the solution of alkyd resin, as obtained from the manufacturer, that was left in a clear glass jar exposed to the light of the laboratory than occurred in the main supply of the resin that had been stored in a gallon container in a dark cabinet. The absorbance of the trough at 264 nm. did not change as much on storage as did the peak at 276 nm. Refrigerated storage in a full container under a blanket of inert gas, such as nitrogen, would prevent such changes.

The protection afforded by the inert atmosphere was demonstrated by tests in which part of the solution of alkyd resin used in the study reported here was stored under an atmosphere of nitrogen for 50 days in a refrigerator. The procedure recommended subsequently was used on samples taken before and after the 50-day storage period. Absorbances for 0.03 percent solutions at 276 and 264 nm. differed by 0.000 and 0.005 absorbance unit, which is well within one standard deviation for replicate tests as given in table 1.

Recommended Procedure

The recommended procedure for determining whether a paint vehicle has been adulterated is described in the following paragraphs.

Remove the pigment from the paint by centrifuging and place the separated vehicle in a well-stoppered amber bottle. Determine the nonvolatile solids content of the vehicle by the procedure described in method 4041 of Federal Test Method Standard No. 141 (7). Weigh by difference in the following manner: Pour several grams of paint vehicle into a small screw-cap dropping bottle equipped with a medicine dropper. Weigh the sealed bottle and contents. Transfer about 1 gram of the vehicle to a weighed aluminum dish, quickly replace the cap, and reweigh the dropping bottle. Avoid sucking the vehicle up into the rubber bulb of the dropper.

Heat a water bath of appropriate size on a variable-control hotplate to a temperature of 56° to 60° C. Weigh by difference from a dropping bottle, a sufficient amount of the paint vehicle to provide about 0.4 gram of nonvolatile solids into a 100-ml. standard-taper, round-bottom Pyrex flask. Stopper the flask immediately.

Set up a rotary-flask evaporator without any lubrication on any of the ground glass joints. The rubber O-ring occasionally needs to be lubricated with a few drops of high-vacuum pump oil. Provide water connections to cool the condenser. Connect the outer jacket with pressure tubing to the stopcock of an unopened vacuum line. Remove the stopper from the flask and attach the flask to the evaporator; hold it in place with the clip provided. Close the glass stopcock at the top of the evaporator. Turn on the vacuum connection and start the rotation of the flask. A vacuum of about 560 mm. of mercury, or more, is satisfactory (absolute pressure of 200 mm., of mercury or less). Place the water bath and hotplate under the flask. Lower the flask until it is about one-half immersed in the water but so that it does not scrape the bottom of the water bath. Start timing at this point; keep the temperature of the bath between 56° and 60° C. If the temperature reaches 60° C. it can be reduced quickly by pouring in a little cold water.

At the conclusion of 20 minutes all of the solvent from the test sample should be evaporated. Then dissolve the dried alkyd resin in cyclohexane. A convenient way to do this is to pour approximately 45 ml. of spectral grade cyclohexane into a 50-ml. beaker and transfer about 4.5 ml. of this into a 5-ml. beaker. At this point turn off the vacuum, remove the pressure tubing from the vacuum line, and

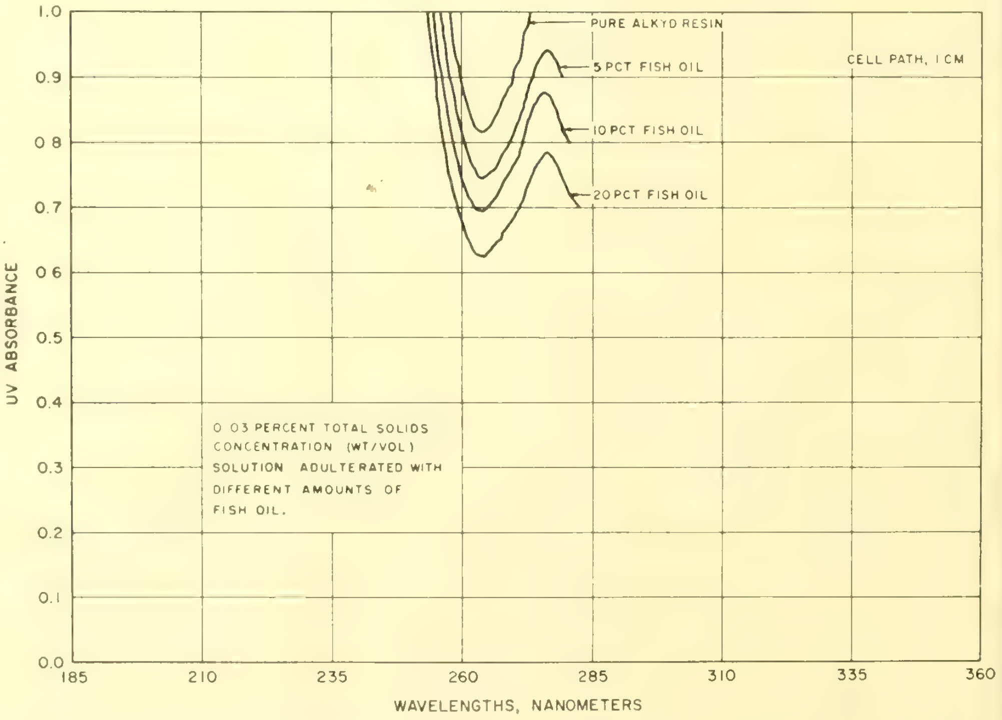

Figure 12.—Spectral absorbance near 264 nm. of alkyd resin adulterated with fish oil.

then reopen the vacuum line. Immediately turn the top ground glass stopcock of the evaporator so that it is open to the air. Hold the 5-ml. beaker under the glass inlet tube and with the other hand push the end of the pressure tubing up against the vacuum-line stopcock opening. This causes the cyclohexane in the 5-ml. beaker to be sucked through the center glass tube and into the evaporating flask. Repeat this procedure until the contents of the 50-ml. beaker have been transferred to the evaporating flask. Then close the ground glass stopcock at the top of the evaporator. Shut off the vacuum line stopcock and leave the pressure tubing disconnected. Continue rotating the flask in the water for 5 minutes to effect complete solution of the residue in the flask. Any convenient procedure for admitting the cyclohexane is acceptable as long as the dried resin is kept under inert conditions insofar as possible.

Table 1.—Repeatability of absorbance measurements made on alkyd resin solution [1]

Absorbance measurements	at 276 nm.			at 264 nm.		at 225 nm.
Concentration, percent [2]	0.03	0.01	0.005	0.03	0.01	0.005
Replicate runs:						
1	1.055	0.353	0.172	0.818	0.276	1.058
2	1.043	0.347	0.174	0.805	0.270	1.069
3	1.032	0.345	0.176	0.798	0.267	1.063
4	1.046	0.350	0.175	0.812	0.274	1.068
5	1.033	0.345	0.186	0.798	0.271	1.094
6	1.017	0.338	0.167	0.781	0.264	1.011
7	1.016	0.337	0.172	0.782	0.263	1.047
8	1.015	0.334	0.168	0.786	0.263	1.039
Average	1.032	0.344	0.174	0.798	0.268	1.056
Range	0.040	0.019	0.019	0.037	0.013	0.083
Standard deviation	0.014	0.007	0.006	0.014	0.005	0.023
Coefficient of variation, percent	1.35	2.04	3.44	1.75	1.87	2.18

[1] In spectral grade cyclohexane using a 1-cm. cell.
[2] Weight per volume basis of alkyd resin in solution.

At the conclusion of the 5-minute period, raise the evaporator so that the water bath and hotplate can be removed. Dry the flask, stop the rotation, and lift the center tube of the evaporator so that it no longer protrudes into the flask. Carefully remove the flask and its contents.

Quantitatively transfer the contents of the flask to a 100-ml. volumetric flask; rinse the evaporator flask well with cyclohexane and collect the rinsings. Repeat the washing procedure until the contents of the volumetric flask are slightly below the mark. After the flask has stood a few minutes and cooled to room temperature, carefully add cyclohexane to the volumetric flask until the contents reach the 100-ml. mark. Make final adjustments dropwise by means of a small pipet—do not use a rubber bulb on the pipet. Mix the contents of the volumetric flask thoroughly.

From this stock solution, appropriate concentrations can be made by putting aliquot parts into 25-ml. volumetric flasks and diluting these to the mark. A convenient equation to use in calculating the milliliters of aliquot to be taken is given in the following example. To make a 0.03 percent solution use:

$$\frac{0.03\times 25}{\text{Weight of sample}\times\text{nonvolatile fraction}}=\text{milliliters of stock solution}$$

For other concentrations, substitute the percent of concentration desired for the figure 0.03. A 2-ml. pipet graduated to 0.01 ml. is convenient for transferring. To prepare a solution as dilute as 0.001 percent, it is advisable to transfer a 2.50-ml. aliquot of a 0.01 percent solution to a 25-ml. volumetric flask and dilute to the mark. It is well to estimate to the nearest 0.001 ml. when using the pipet in order to insure an accuracy in measurement of about one part in 100.

Obtain absorbance curves with the spectrophotometer using a scanning rate of about 20 nanometers per minute. Before each run, establish a baseline that shows the relative absorbances of the two cells containing only cyclohexane. This procedure provides evidence of cleanliness of cells and how well they are matched. In the work reported here, the alkyd resin coating on the cells could not be removed completely with any of the organic solvents or detergents that were tried. The cells, therefore, were cleaned by immersing them in freshly prepared chromic acid cleaning solution after each day's use (30-ml. beakers were used). After being rinsed thoroughly with distilled water, the cells were dried at 105° C. The instructions for operating the spectrophotometer are contained in the instruction manual provided by the manufacturer of the instrument (*8*).

A convenient method for pouring solutions into the 1-centimeter ultraviolet cells is to transfer about 3 ml. of each solution into a 5-ml. beaker and then to the cell. After the absorbance has been measured, if the solution to be discarded is poured out of the cell, it tends to run over the clear outside surface. Wiping or washing these sides during a test leaves them in such a condition as to cast doubt on subsequent absorbance measurements. This difficulty can be avoided in the following manner: Pull out a length of glass tubing to a small diameter and fire polish at both ends. Connect the tubing to a vacuum line through a safety flask. With the vacuum turned on, the smaller diameter part of the bent tube is very effective in removing solutions from the cells. Lower the tube down one of the frosted sides of the cell but do not allow it to touch the clear-glass windows through which the ultraviolet radiation passes.

Use this described procedure on the original road test sample of traffic paint submitted and on samples from the purchased lots of paint. It is desirable that the final spectral test solution be such that the more pronounced spectral peaks and troughs will occur at absorbance values near or below 1. The purchaser can be assured that the purchased paint vehicle has not been adulterated if the following criteria are met: (1) The pronounced spectral peaks and troughs for identical concentrations and wavelengths are within 2 to 3 parts in 100 of the same absorbance value. (2) Qualitative examination of the ultraviolet absorption patterns indicates the comparison spectrum is not shifted significantly in wavelength or otherwise different. If these criteria are not met, it can be assumed that the purchased lot has been adulterated. It is possible to complete the solvent removal and obtain the patterns for two determinations within an 8-hour day, including time required for cleanup.

Conclusions and Applications

From the results obtained in the research reported here, it is concluded that the adulteration of traffic paint vehicle solids can be detected by the absorption of ultraviolet radiation in the range 210 to 360 nm. A rotating-flask evaporator operating under line vacuum of 560 mm. of mercury can remove volatile solvents without noticeably affecting the ultraviolet pattern of an alkyd resin. Cyclohexane was determined to be satisfactory for ultraviolet spectral analysis because it adequately redissolved the vehicle solids and also had satisfactory ultraviolet absorption characteristics. On the basis of materials investigated, it is concluded that adulteration in amounts as low as 2 to 5 percent by weight of the vehicle solids can be detected.

The techniques described and principles involved seem to be applicable to vehicles and adulterants other than those included in this report. Modifications of the method developed would also seem to be applicable to paint vehicles in protective coatings for steel. Even when composition is specified, the ultraviolet spectral characteristics may be useful as a rapid method for determining compliance with specifications. Some of these other applications of ultraviolet spectral analysis are being investigated by the authors.

ACKNOWLEDGMENT

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(*Continued on p. 266*)

Effects of Volume Controls on Freeway Traffic Flow—A Theoretical Analysis

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

Reported by EDMUND A. HODGKINS,[1] Highway Research Engineer, Traffic Systems Division

Traffic congestion on urban freeways is a major concern of traffic engineers faced with the problem of maintaining traffic flow, and in this article several methods for reducing peak period congestion are discussed. Methods are presented for calculating metering rates for certain ramp metering systems, queue lengths, delays, and lengths of congestion period. A hypothetical ramp metering problem illustrates the benefits to be gained by eliminating congestion from the freeways.

There are several methods of regulating freeway demand, and their use either individually or in combination can be an effective tool for transportation engineers. Ramp metering techniques have been effective in decreasing travel time, increasing average speeds, and reducing the number of accidents. Of the several methods of ramp metering, ramp closure and fixed time metering have been useful in eliminating recurring congestion. Dynamic ramp metering may eventually provide a major contribution to the elimination of congestion. Current systems have reduced congestion a great deal but have not completely eliminated it. Economic constraints may be effective in regulating demand, but their effect on traffic flow has not been determined and they could adversely effect economic development of the areas where they are used. Dynamic diversion of traffic—monitoring traffic conditions and informing the driver of alternative congestion-free routes—can prevent congestion in sections of a network and improve overall system utilization. Even the staggering of hours can be effective in regulating demand under certain circumstances.

Introduction

PEAK PERIOD CONGESTION on urban freeways is a problem that is growing to alarming proportions. Various methods of controlling demand on a freeway, a freeway system, or the whole urban area street-highway system, have been suggested by those concerned with the problem, but the methods currently being emphasized are freeway traffic surveillance and control. The primary objective of most surveillance and control projects is the reduction or elimination of congestion, and the methods used are designed to limit the flow of traffic into a section of freeway (ramp metering) or to distribute traffic throughout a system so that all links in a network operate at or below capacity. Although there are several freeway control projects now in operation throughout the country, none has yet succeeded in eliminating freeway congestion; however, congestion has been greatly reduced at some locations.

Several methods of reducing peak period congestion on freeways are discussed in this article and include ramp metering (ramp closure, fixed time ramp metering, dynamic ramp metering, gap availability metering, and system demand capacity metering); dynamic diversion of traffic; the concept of staggered hours; and the concept of economic constraint. In addition to the discussion of different methods of control, methods are formulated for calculating (1) metering rates for time-based metering systems, (2) maximum queue length, (3) total delay, and (4) length of congested period or ramp-metering period. These methods are then applied to a simple hypothetical ramp metering situation and an economy analysis is performed. The hypothetical ramp metering situation serves as a conservative example of the benefits to be obtained from eliminating freeway congestion.

In order to provide a facility that operates smoothly, it is necessary to design a system free of traffic bottlenecks. A bottleneck has been defined as a location at which the demand can exceed the capacity, *(1)*[2] or the point at

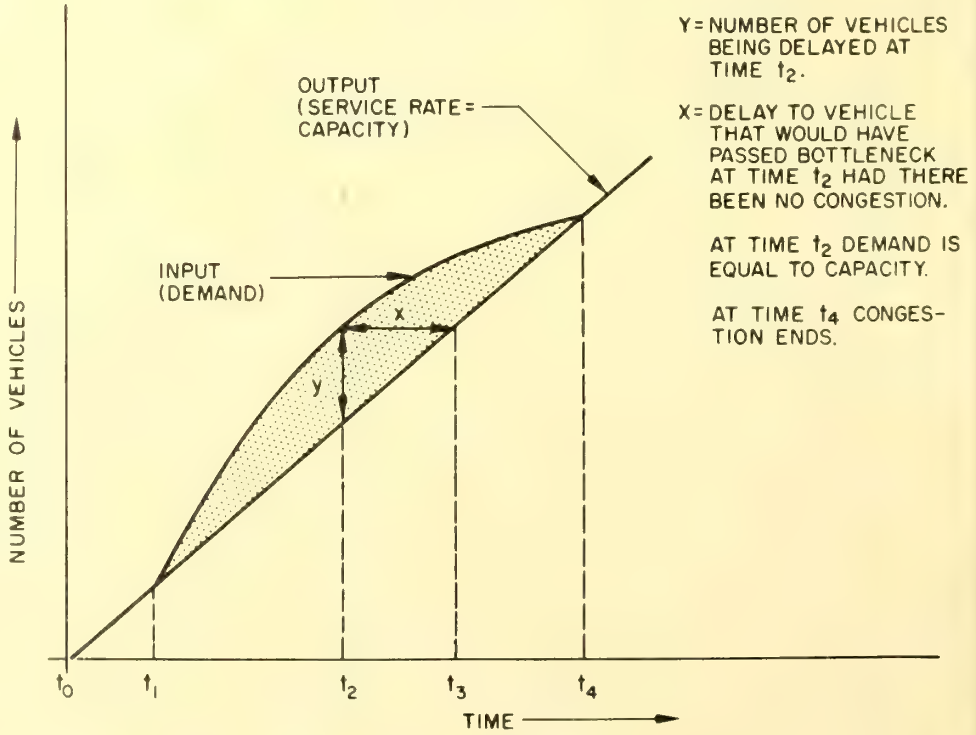

Figure 1.—Flow rate at a bottleneck.

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[1] Mr. Hodgkins is the leader of the Stream Flow Task Group, a Public Roads research group concerned with the formulation and evaluation of traffic flow theory, the development of area traffic surveillance and control systems, and the evaluation and development of automatic highway systems.

[2] References indicated by italic numbers in parentheses are listed on page 266.

which the capacity decreases and/or the demand increases. The capacity of most highway facilities is not uniform. Conditions such as adverse grades, reduction in the number of lanes, short radius curves, heavy weaving movements, or heavy ramp movements (either on-ramp or off-ramp) will tend to reduce the capacity at given points on the facility. If the volume of traffic approaching a point of restricted capacity is larger than the capacity of that point, congestion will result.

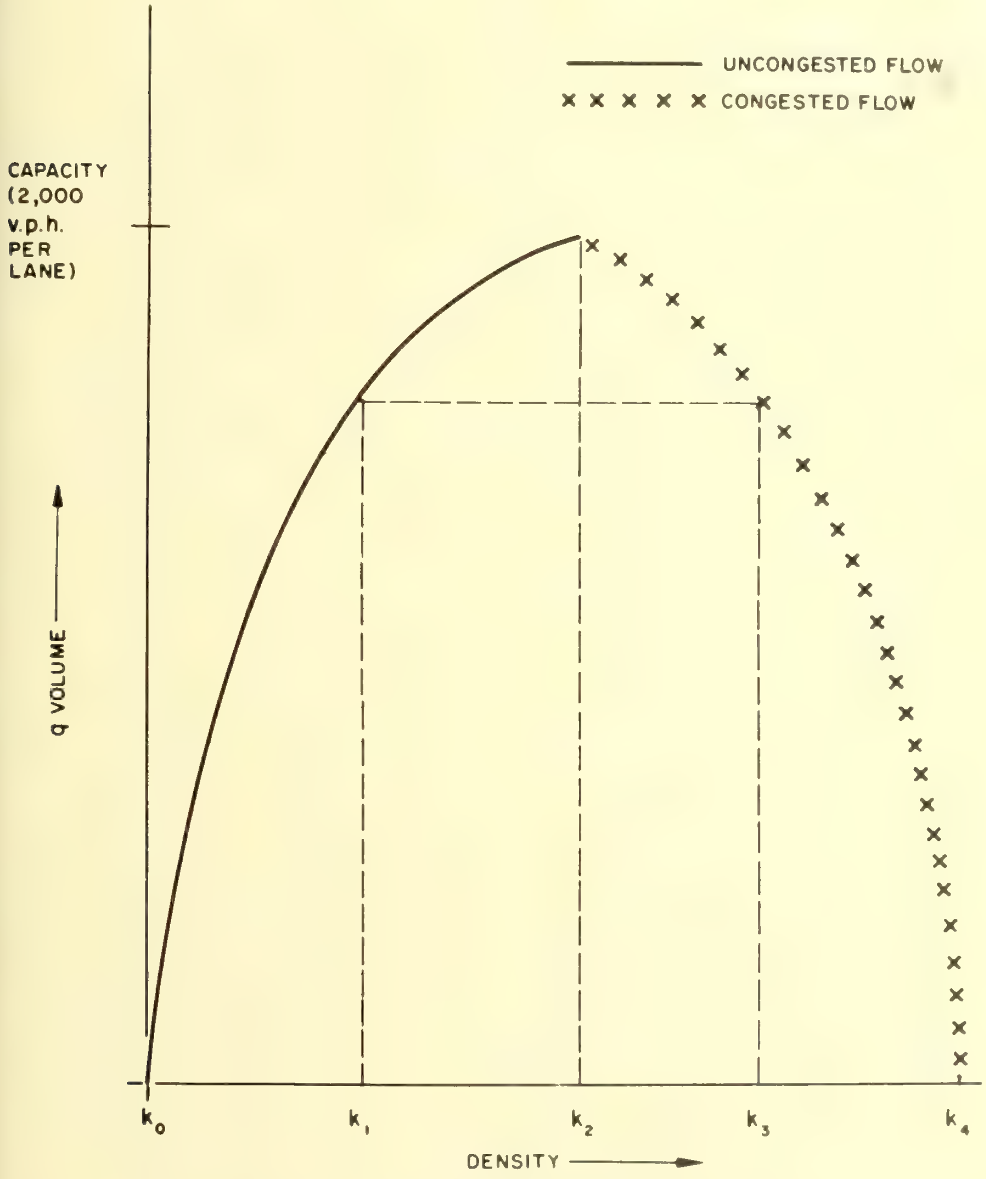

Figure 2.—Typical volume-density relationship.

The flow rate at a bottleneck is illustrated in figure 1. As shown, from time t_0 to time t_1, the input rate at this point on the highway is equal to the output rate. For this example the flow rate from time t_0 to t_1 equals the capacity of the facility. At time t_1 the demand (input) exceeds the capacity; the output or service rate cannot exceed capacity, therefore, a queue begins to form. At time t_2 the input or demand is less than capacity, but the congestion lasts until time t_4. The shaded area represents total delay. Any abscissa (x) represents delay to an individual vehicle, and any ordinate (y) represents the number of vehicles being delayed at a point in time. To eliminate the delay, it is necessary either to increase the capacity at the bottleneck or to decrease the rate of flow into the bottleneck. Most of the methods of volume control discussed in this paper are those designed to limit the rate of flow entering the system.

An important factor to be considered when discussing bottleneck capacity is the actual reduction in effective capacity that occurs when the freeway becomes congested. In figure 1 it is assumed that there is no reduction in capacity. Reduction in effective capacity can be partly attributed to a decrease in volume when the density in vehicles per mile exceeds a certain critical value (in the range of 50 to 60 vehicles per lane mile), as shown in figure 2.

The mathematical expression for the volume of a highway lane is:

$$q = kv$$

Where,

q= the flow rate in vehicles per hour.
k= the density in vehicles per mile.
v= the speed in miles per hour.

The effect of the terms of this equation is depicted in figure 2 in which it is shown that by decreasing the density from k_3 to k_1, an equivalent volume can be moved at higher speed and that by decreasing the density from k_3 to k_2 a larger volume can be moved at a higher speed. Density k_2 is the density at capacity in this illustration; when the density is greater than k_2 congestion will occur.

The reduction in effective capacity can be partly attributed to another phenomenon, which is reported in the Highway Capacity Manual 1965 (*2*), on pages 78 and 108. It is noted that, although the normal lane capacity of a multilane highway is 2,000 vehicles per hour, if the traffic flow is stopped, the flow rate leaving the point of interrupted flow usually will not exceed 1,500 vehicles per hour. This is an effective reduction in capacity of 500 vehicles per hour, a reduction which cannot be regained except by the addition of 500 vehicles per hour to the traffic downstream from the point of interruption. This phenomenon is illustrated in figure 3.

In figure 4, the effect of an effective reduction in capacity on the cumulative flow over a period of time is compared with the results shown in figure 1, where it is implicitly assumed that there is no effective reduction in capacity. As illustrated, the total delay will be larger, the delay to individual vehicles will be longer, the maximum queue length will be greater, and the freeway will be congested over a longer period of time.

Ramp Metering

For a long time, traffic engineers have thought of metering freeway traffic as a method of effectively controlling inputs to the freeway, thereby providing desirable operating conditions on the freeway system. In theory it is possible to prevent congestion by controlling the vehicular densities in the freeway system. In the introduction to this paper it was shown that the elimination of congestion results in an increase in speed, or a decrease in travel time, on the freeway, and could result in an increase in volume.

In a research report (*3*), David Solomon shows that a reduction in congestion results in fewer accidents. In other research studies, it has been shown that a reduction in travel time, an increase in driver comfort, and an increase in the effective volume carrying capability of the facility can also be realized from reduced congestion. Ramp metering experiments at the Chicago Area Expressway Surveillance project have produced a 25-percent increase in volume, a sharp reduction in peak-period congestion, and a significant reduction in the accident rate.

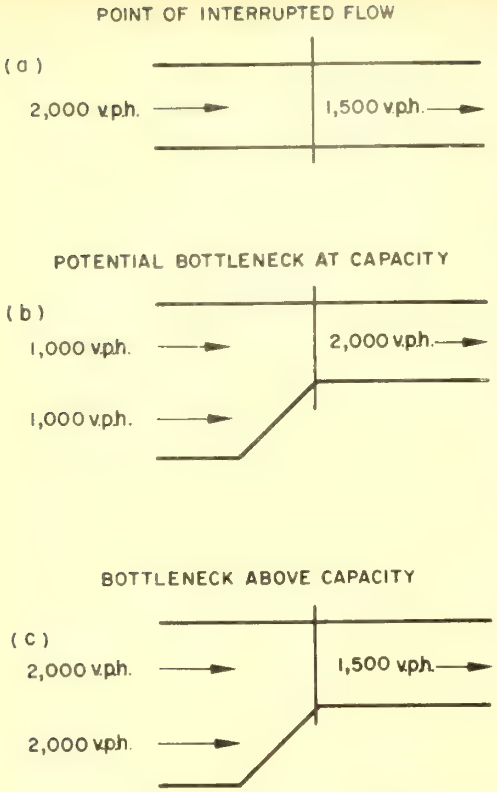

Figure 3.—Effects of interrupting the flow of traffic

To be completely effective and provide the most efficient use of the freeway system all entrance ramps should be controlled and all exit ramps should be monitored. The several methods of ramp metering include ramp closure, fixed time ramp metering, dynamic ramp metering, gap availability metering, and system demand-capacity metering. Each of these methods is discussed briefly in the following paragraphs.

Ramp closure (*4*) may be thought of either as a change in geometrics (elimination of a ramp) when a ramp is closed for the duration of the peak period or, as a special case of metering, when closure periods range from 5 to 15 minutes at a time during peak periods. Ramp closure can inconvenience drivers who usually use a ramp, and care must be taken to ensure that alternate routes are available to those drivers affected.

Fixed-time ramp metering is based on the premise that ramp and freeway demand is relatively predictable from day to day, and no attempt is made to account for unusual events such as traffic accidents. By using information gathered in origin and destination surveys, ramp and freeway demand are estimated, and signals are installed on the ramps to feed ramp traffic into the freeway flow at predetermined flow rates during peak periods. This type of system operation is based on historical data; it does not respond to traffic flow conditions on the freeway.

Dynamic ramp metering requires an analytic definition of freeway traffic flow to be used effectively. By measuring certain traffic-flow parameters at strategic points on the freeway and on the on-ramp, metering rates are varied incrementally using previously derived relations between the flow rate and the parameter being measured on the freeway. For example, metering rates on the Eisenhower Expressway in Chicago are based on a relation between lane 2 (middle lane) occupancy[3] upstream from the ramp and the freeway flow rate (see table 1), and metering rates on the Gulf Freeway in Houston are based on a relation between the speed in lane 1 (right shoulder lane) downstream from the nose of the ramp and the freeway flow rate. Queuing on the ramp is measured by different methods, and attempts are made to minimize storage on the ramps and frontage roads.

Both *gap availability metering* and *system demand capacity metering* are forms of dynamic ramp metering, but their application is slightly different from the general form of dynamic ramp metering in which the metering rate is based on a relation between a traffic flow parameter, measured on the freeway, and the freeway flow rate.

In *gap availability metering* the spacing in the traffic stream in lane 1 (right shoulder lane) upstream from the ramp is measured, and the signal at the ramp is programed to allow one, two, three or more vehicles to enter a gap in the stream that is based on a predetermined acceptable gap per vehicle. Occasionally vehicles may switch lanes and fill the gap prior to the arrival of the gap at the nose of the ramp, but a detector at the nose of the ramp indicates whether a vehicle or vehicles have managed to enter the freeway.

All of these metering systems except system demand capacity metering can be used at individual ramp locations with varying degrees of success. Generally, it is more practical to meter a series of ramps or all of the ramps in a controlled area to maximize the probability of eliminating congestion. *System demand capacity metering,* however, is an analytic method for metering the entire system rather than just individual ramps or a series of ramps. System demand capacity metering might be visualized in its simplest form as measuring the inputs and outputs of a system and controlling entrance to the system so that a predetermined optimum density can be maintained.

Research is now being conducted using combinations of all these ramp metering methods to provide more flexible, optimum metering rates in an effort to approach relatively high-speed, near-capacity operation.

Ramp metering has progressed a great deal in the last 6 years, evolving from a concept and advancing to the beginnings of an operational practicality. The ramp metering systems that have been designed to date are admittedly crude; but when a better understanding of the functional relationships of traffic flow has been achieved, the application of these analytic descriptions of traffic flow to ramp metering and to other types of control will provide greater sophistication in traffic surveillance and control, and current experiments in ramp metering will be recognized as highly valuable contributions to the total knowledge that enabled man to control his driving environment effectively.

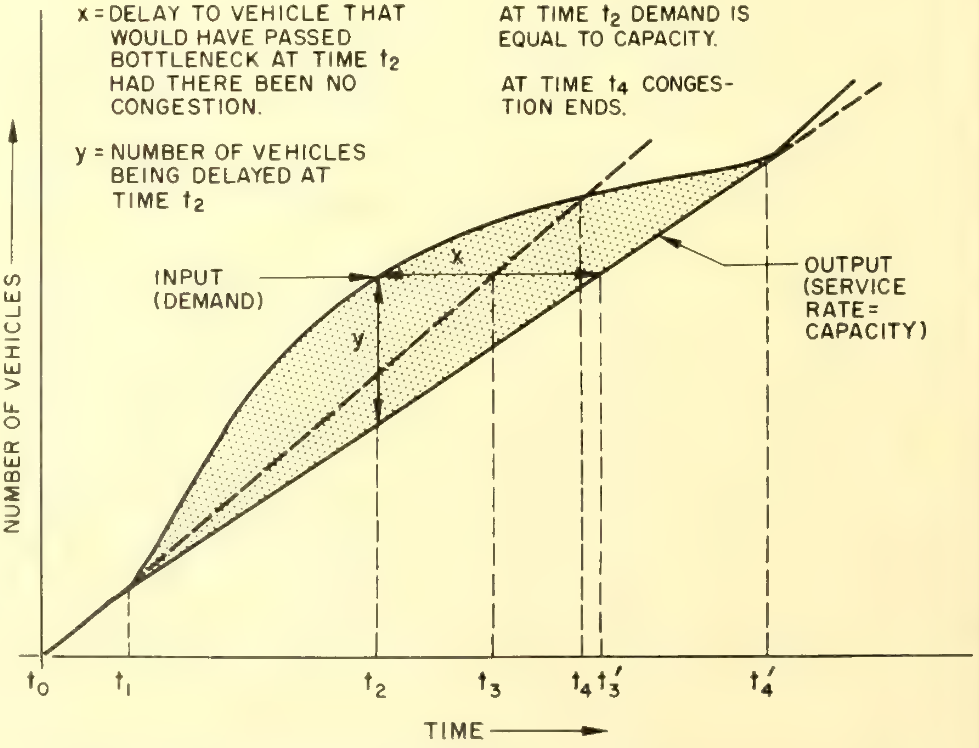

Figure 4.—Effect of a substantial reduction in capacity on the flow rate at a bottleneck.

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[3] Occupancy is defined as the percentage of time that the space under a detector unit is occupied by a vehicle on the roadway.

Table 1.—Metering rates for various occupancy levels

Lane-2 occupancy	Metering rate
Percent	*v.p.m.*
15	12.0
16	12.0
17	12.0
18	8.5
19	8.5
20	8.5
21	7.5
22	6.5
23	5.0
24	5.0

Dynamic Diversion of Traffic

One concept for reducing system demand on links of freeway having insufficient capacity is dynamic traffic assignment—the assignment of traffic based on the operating conditions on any of the links of the freeway-highway network. By measuring traffic flow parameters on several different links of the freeway-highway network it should be possible to determine when the demand is about to exceed the service rate of any particular link. When this condition exists a computer system will activate the proper communication devices to inform the driver of all alternate routes which will provide better service and minimize his overall travel time in his home-to-work or work-to-home trip. To effectively use dynamic diversion of traffic, it is necessary to measure those traffic flow parameters that will adequately predict the current state of traffic dynamics at any point in time. In a sense dynamic diversion of traffic does not really reduce the system demand, but it does provide the means for optimizing use of the system to minimize travel time and to offset congestion. If the demands on the system as a whole exceed the system capacity, then it becomes necessary to reduce the inputs into the system. Traffic-flow models to prescribe the dynamic diversion of traffic throughout any freeway-highway network can be derived by using the same linear programing techniques that are currently used in assembling traffic assignment and traffic distribution models for highway planning purposes. It is possible that with a better knowledge of the economic and traffic flow parameters, models now used for traffic assignment and traffic distribution can be easily modified for use in programing the dynamic diversion of traffic.

An example of a crude though often effective technique of diverting traffic based on existing traffic-flow conditions is the traffic reporter in a helicopter communicating with the driver through local radio stations. More complicated and effective techniques for accomplishing the dynamic diversion of traffic would require an extensive system of detector units to measure the traffiic-flow parameters, and a system to communicate these measurements to a central computer complex that would continually record the state of the traffic. This is essentially what has been done in Toronto and what is being done in San Jose and New York City in an attempt to provide a coordinated system of signals throughout the city-street network. Dynamic diversion of traffic, however, would be applied to the freeway-highway system; a central computer would analyze and record the effective traffic flow parameters and provide appropriate information to drivers so that they could follow the optimum route to their destination either by choice or by enforcement of more stringent routing controls.

Dynamic diversion of traffic has been experimented with in a very limited way on the John C. Lodge Expressway, Detroit, Mich.—the National Proving Ground for Freeway Surveillance, Control, and Electronic Traffic Aids. When an accident or heavy congestion occurs on the freeway, red X's over each lane light up and a green arrow at the exit ramps begins flashing. Drivers on the John C. Lodge Expressway have learned that this informational display indicates a blocked expressway that they should leave it at the next exit.

In Chicago, informational signs have been placed on city streets to alert the driver approaching an on-ramp of conditions on the Eisenhower Expressway and adjacent ramps. Changeable color arrows are used to indicate the traffic flow condition on the expressway and ramp. A red arrow indicates congestion has set in, an amber arrow indicates slow moving traffic and impending congestion, and a green arrow indicates free flow. Preliminary studies indicate that very little diversion can be attributed to these signs.

Other Methods of Volume Control That Have Been Proposed

In addition to ramp metering and dynamic diversion, several other methods of effectively reducing demand to a freeway system have been proposed. Two of these methods are staggered hours and economic constraints.

The concept of staggered hours

It is felt that by having people start work at different times of the day, the demand for use of the freeway-highway system could be spread over a longer period of time, thus relieving congestion and all its concomitant ills. However, there are several disadvantages to using the concept of staggered hours:

- Although it is no longer feasible from the standpoint of highway use, it may be essential in some areas of employment for everyone to work the same hours. Therefore, in staggering hours between different employees, it is necessary to account for the interrelations and interactions that take place during any normal work day.
- Staggered work hours tend to work against other factors that contribute to a more favorable demand-capacity relation; for example, the staggering of hours could adversely affect the formation of carpools and conceivably result in an increase in total vehicle miles of travel and a decrease in average vehicle occupancy.
- It is impossible to account for all vehicles in the system; traffic for shopping, business, school, social trips, etc., cannot be effectively regulated by staggering hours.
- The staggering of hours is directly affected by network configuration and land use.

The University of Arizona conducted a study to formulate a mathematical model for determining the extent to which traffic loads could be distributed over a longer period of time and for establishing the spread of hours needed to effect economies in the level of service (*5*). Results of this study indicated that staggering hours is technically feasible, but that the economic cost factors needed to use the model effectively were not completely known at the time. In general, it would seem that, to spread the demand over a long enough period to cause a significant reduction in the demand, and thus enable the highway system to operate at or below its capacity, would cause too long a spread in demand—perhaps too long in any single working day to be acceptable in a larger city. In medium-to-large cities, staggering hours would have to take place over a 2- to 3-hour period to accomplish significant reductions in demand during the peak hours.

The staggering of hours alone may not be one of the better methods of adjusting demand to match capacity, but this concept used in conjunction with dynamic diversion and ramp metering might prove to be an effective tool in regulating traffic flow.

The concept of economic constraint

Economic constraint, as a method for controlling volume and thus regulating demand so that the service rate of the facility is not exceeded, states that demand can be reduced through the imposition of a toll or other user charge that is so high that the user will attempt to find alternate modes of less expensive transportation. One disadvantage of this concept is that it seems to let the rich man use the roadway and force the poor man to use mass transportation facilities or at least reduce his flexibility by forcing him to form carpools.

Several methods have been proposed for imposing these economic constraints. One is simply to placing tollgates at all roadways leading into an urban area; another is to place detector loops in certain critical areas within the city and to require that all vehicles traveling within the city be equipped with meters that would measure the time each vehicle is operated or parked within these critical areas. The payment for this time would be directly proportional to the demand that these areas normally engender during peak periods. Still another method of economic constraint—one less stringent than the two just mentioned—is to provide free mass transportation facilities so that the driver has a choice of spending money to drive his own vehicle or traveling on a transit system (either bus or rail), which he is subsidizing.

All of these economic constraints should cause a reduction in demand for use of a facility or system and, if the constraint is stringent enough, enable the facility to handle flows within its capacity. In the long run, however, any urban area imposing such economic constraints as toll charges or exorbitant parking fees is likely to contribute to the decline of the central urban area and to promote even greater urban area decentralization than has heretofore been experienced.

Development of a Method for Determining Time Based Metering Rates

As stated in the introduction, fixed-time ramp metering is based on the premise that ramp and freeway demand is relatively predictable, and this type of ramp metering can be used to prevent recurring congestion. A procedure to determine the fixed-time ramp-metering rates for a single on-ramp is developed in the following paragraphs. This procedure can be adapted to dynamic ramp-metering situations and, if necessary, extended to account for a series of bottlenecks. For the purposes of this article, however, the additional complexities required to adapt or extend these procedures for several bottlenecks would provide no better illustration of the use of ramp metering or other volume controls than that given for the single ramp.

The following notation will be used for the remainder of this article:

$f(t)$ = freeway demand (vehicles per hour) at the merge point as a function of time

$r(t)$ = ramp demand (vehicles per hour) at the merge point as a function of time

$d(t)$ = the total demand (vehicles per hour) at the merge point as a function of time

C_{m1} = the capacity (vehicles per hour) of the merge point before congestion occurs

C_{m2} = the effective capacity (vehicles per hour) of the merge point after congestion occurs

q_m = the output flow rate (vehicles per hour) at the merge point

$$q_m = d(t) \qquad (1)$$

Where, $d(t) \leq C_{m1}$

$$q_m = C_{m2} \qquad (2)$$

Where, $d(t) > C_{m1}$ and metering is not employed

$$q_m = C_{m1} \qquad (3)$$

Where, $d(t) > C_{m1}$ and metering is employed

$L(t)$ = the length of queue (number of vehicles) at time t

$D(t)$ = the total delay (vehicle-hours) at time t

t_o = the time when $f(t) + r(t) = C_{m1}$

t = the time (hours)

$r^c(t)$ = the ramp metering rate (vehicles per hour) as a function of time

It is obvious that:

$$d(t) = f(t) + r(t) \qquad (4)$$

In order to eliminate congestion at the bottleneck it has been shown that q_m must be less than or equal to C_{m1}.

The objective of ramp metering is to eliminate congestion at the bottleneck. Congestion is defined as the traffic flow condition that occurs when capacity is exceeded; therefore, the following equations can be derived:

$$r^c(t) = C_{m1} - f(t) \qquad (5)$$

Where,

$$f(t) < C_{m1}$$

and

$$q_m = f(t) + r^c(t) = C_{m1} \qquad (6)$$

Where,

$$d(t) > C_{m1}$$

The above procedure allows the freeway to operate at capacity. Freeways have been observed to flow at capacity; that is, 2,000 vehicles per hour per lane, for the entire peak period under the proper operating conditions. Ramp metering ensures that the proper operating conditions are maintained.

Without ramp metering the queue lengths and delays are calculated as follows:

$$L(t) = \int_{t_o}^{t} d(t)dt - \int_{t_o}^{t} C_{m2}dt \qquad (7)$$

$$D(t) = \int_{t_o}^{t} L(t)dt \qquad (8)$$

Equation (7) gives the total number of vehicles in a queue at any point in time. This total should be divided between the freeway and the ramp in a ratio proportional to the respective demands. The same procedure should be used for equation (8).

With ramp metering the queue lengths and delays are calculated as follows:

$$L^c(t) = \int_{t_o}^{t} r(t)dt - \int_{t_o}^{t} r^c(t)dt \qquad (9)$$

$$D^c(t) = \int_{t_o}^{t} L^c(t)dt \qquad (10)$$

The length of the period of congestion (or ramp metering) can be calculated by determining the time t at which the queue length is reduced to zero and then subtracting t_o, the time at which the congestion began.

This procedure allows the freeway to keep flowing at capacity at the expense of the vehicles using the ramp. In fact use of this procedure will result in ramp closure during those periods when the freeway is operating at capacity. It must be assumed that there is either ample storage at the ramp or an alternate route available for use by the ramp vehicles. If an alternate route is available, the delay to the vehicles normally using the ramp will, in all likelihood, be less than if these vehicles were to be stored in queues at the ramp. In the calculations and analysis that follow it is assumed that no alternate route is available, but that there is ample space for storage, thus the ramp vehicles will be penalized to allow optimum operation of the system as a whole.

Sample Calculations

The problem

In the preceding section the analytic procedures for calculating the metering rate, length of congestion time, length of queue, and tota delay were developed. In the following paragraphs a solution of a simplified metering situation is completed to illustrate the use of these procedures.

The metering location of this example is a single on-ramp to a single-lane freeway. The following equations supply the information necessary to solve this problem:

$$f_1(t) = 1,400 + 600t \qquad 0 \leq t \leq 1 \qquad (11)$$

$$f_2(t) = 2,600 - 600t \qquad 1 \leq t \leq 2 \qquad (12)$$

$$f_3(t) = 1,400 \qquad 2 \leq t \leq 24 \qquad (13)$$

$$r_1(t) = 200 + 600t \qquad 0 \leq t \leq 1 \qquad (14)$$

$$r_2(t) = 1,400 - 600t \qquad 1 \leq t \leq 2 \qquad (15)$$

$$r_3(t) = 200 \qquad 2 \leq t \leq 24 \qquad (16)$$

$$d_1(t) = 1,600 + 1,200t \qquad 0 \leq t \leq 1 \qquad (17)$$

$$d_2(t) = 4,000 - 1,200t \qquad 1 \leq t \leq 2 \qquad (18)$$

$$d_3(t) = 1,600 \qquad 2 \leq t \leq 24 \qquad (19)$$

C_{m1} = 2,000 vehicles per hour

C_{m2} = 1,800 vehicles per hour

The problem is to find the following:

- The metering rate at which the freeway flow will not exceed C_{m1} (2,000 vehicles per hour).
- The maximum length of queue under uncontrolled conditions.
- The maximum length of queue under controlled conditions.
- The total delay under uncontrolled conditions.
- The total delay under controlled conditions.
- The length of time congestion lasts under uncontrolled conditions.
- The length of time metering is required under controlled conditions.

Figure 5 is a graphical representation of the problem and its solution.

Calculation of metering rate

The metering rate can be calculated using equation (5) in the previous section.

$r^c(t) = C_{m1} - f(t)$

$r_1^c(t) = 2,000 - (1,400 + 600t) = 600\ (1-t)$

$$t_o \leq t \leq 1 \qquad (20)$$

$r_2^c(t) = 2,000 - (2,600 - 600t) = 600\ (t-1)$

$$1 \leq t \leq 2 \qquad (21)$$

$r_3^c(\mathrm{t}) = 2,000 - 1,400 = 600\ v.p.h.$

$$2 \leq t \leq t_3 \qquad (22)$$

Where,

t_o = the time at which the total demand is first equal to 2,000 v.p.h.

and

t_3 = the time at which the queue length is reduced to zero.

Using equation (17) t_o can be calculated:

$$2,000 = 1,600 + 1,200t_o$$

$$t_o = 1/3 \text{ hour}$$

Calculation of maximum queue length

The queue length for uncontrolled conditions can be calculated using equation (7) of the previous section.

$$L_1(t)=\int_{t=1/3}^{t}(1{,}600+1{,}200t)\,dt-\int_{t=1/3}^{t}1{,}800\,dt$$

For any value of time in the given interval the following equation will give queue length:

$$L_1(t)=600t^2-200t \qquad \tfrac{1}{3}\leq t\leq 1 \qquad (23)$$

Because the function for flow rate is constructed from three separate continuous functions in time, the queue length at $t=1$ in equation (23) must be added to the calculation for queue length for the time period from 1 to 2 in figure 5. Thus

$$L_2(t)=400+\int_{t=1}^{t}(4{,}000-1{,}200t)\,dt-\int_{t=1}^{t}1{,}800\,dt$$

$$L_2(t)=2{,}200t-600t^2-1{,}200 \qquad 1\leq t\leq 2 \qquad (24)$$

Similarly

$$L_3(t)=1{,}200-200t \qquad 2\leq t\leq t_4 \qquad (25)$$

Using equation (25) t_4 can be calculated:

$$L_3(t)=0=1{,}200-200t_4$$
$$t_4=6 \text{ hours}$$

Using equations (18) and (24) the maximum queue length can be calculated. The maximum queue length occurs when

$$d(t)=C_{m2} \text{ at } t_2>t_o$$
$$C_{m2}=1{,}800=d_2(t)=4{,}000-1{,}200\,t_2$$
$$t_2=\frac{2{,}200}{1{,}200}=1\tfrac{5}{6} \text{ hours}$$

Using equation (24) the maximum queue length can be calculated to be:

$$L_2(t)=2{,}200\,t_2-600(t_2)^2-1{,}200$$
$$=2{,}200\left(\frac{11}{6}\right)-600\left(\frac{121}{36}\right)-1{,}200$$

Maximum Queue Length = 816 vehicles

This represents 682 vehicles on the freeway and 134 vehicles on the ramp.

The queue length for controlled conditions can be calculated using equation (9):

$$L^c(t)=\int_{t_o}^{t}r(t)\,dt-\int_{t_o}^{t}r^c(t)\,dt$$

$$L^c_1(t)=\int_{t_o}^{t}(200+600t)\,dt-\int_{t_o}^{t}(600-600t)\,dt$$

$$L^c_1(t)=600t^2-400t+67 \qquad \tfrac{1}{3}\leq t\leq 1 \qquad (26)$$

As three separate continuous functions are used to describe flow rate, the queue length at $t=1$ must be added to the calculation for queue length for the time period from 1 to 2 in figure 5. Thus

$$L^c_2(t)=267+\int_{t=1}^{t}(1{,}400-600\,t)\,dt-\int_{t=1}^{t}(600\,t-600)\,dt$$

$$L^c_2(t)=2{,}000t-600t^2-1{,}133 \qquad 1\leq t\leq 2 \qquad (27)$$

Similarly the calculation of queue length for the time period from 2 to t_3 results in the following equation:

$$L^c_3(t)=1{,}267-400t \qquad 2\leq t\leq t_3 \qquad (28)$$

From the above equation it can be shown that $t_3=3\tfrac{1}{6}$ hours.

The maximum queue length occurs when $r(t)=r^c(t)$ at $t_1>t_0$. Using equations (15) and (21), t_1 can be calculated:

$$1{,}400-600t_1=600t_1-600$$
$$2{,}000=1{,}200t_1$$
$$t_1=1\tfrac{2}{3} \text{ hours}$$

The maximum queue length which occurs at t_1 can be calculated from equation (27).

Maximum queue length $=2{,}000(\tfrac{5}{3})-600(\tfrac{25}{9})-1{,}133=534$ vehicles

These vehicles will be stored at the ramp.

Calculation of total delay

The total delay can be calculated using the equation:

$$D(t)=\int_{t_o}^{t}L(t)\,dt$$

It must be remembered that three separate continuous functions were used to describe flow rate; the calculations for cumulative delay result in the following equations for uncontrolled conditions:

$$D_1(t)=200t^3-100t^2+4 \qquad \tfrac{1}{3}\leq t\leq 1 \qquad (29)$$

$$D_2(t)=1{,}100t^2-200t^3-1{,}200t+404 \qquad 1\leq t\leq 2 \qquad (30)$$

$$D_3(t)=1{,}200t-100t^2-1{,}196 \qquad 2\leq t\leq t_4 \qquad (31)$$

To calculate the total delay t_4 must be substituted for t in equation (31). The total delay in the uncontrolled situation is 2,404 vehicle hours.

The calculations for total delay for controlled flow result in the following equations:

$$D^c_1(t)=200t^3-200t^2+67t-7 \qquad \tfrac{1}{3}\leq t\leq 1 \qquad (32)$$

$$D^c_1(t)=393-1{,}133t+1{,}000t^2-200t^3 \qquad 1\leq t\leq 2 \qquad (33)$$

$$D^c_3(t)=1{,}267t-200t^2-1{,}207 \qquad 2\leq t\leq 3\tfrac{1}{6} \qquad (34)$$

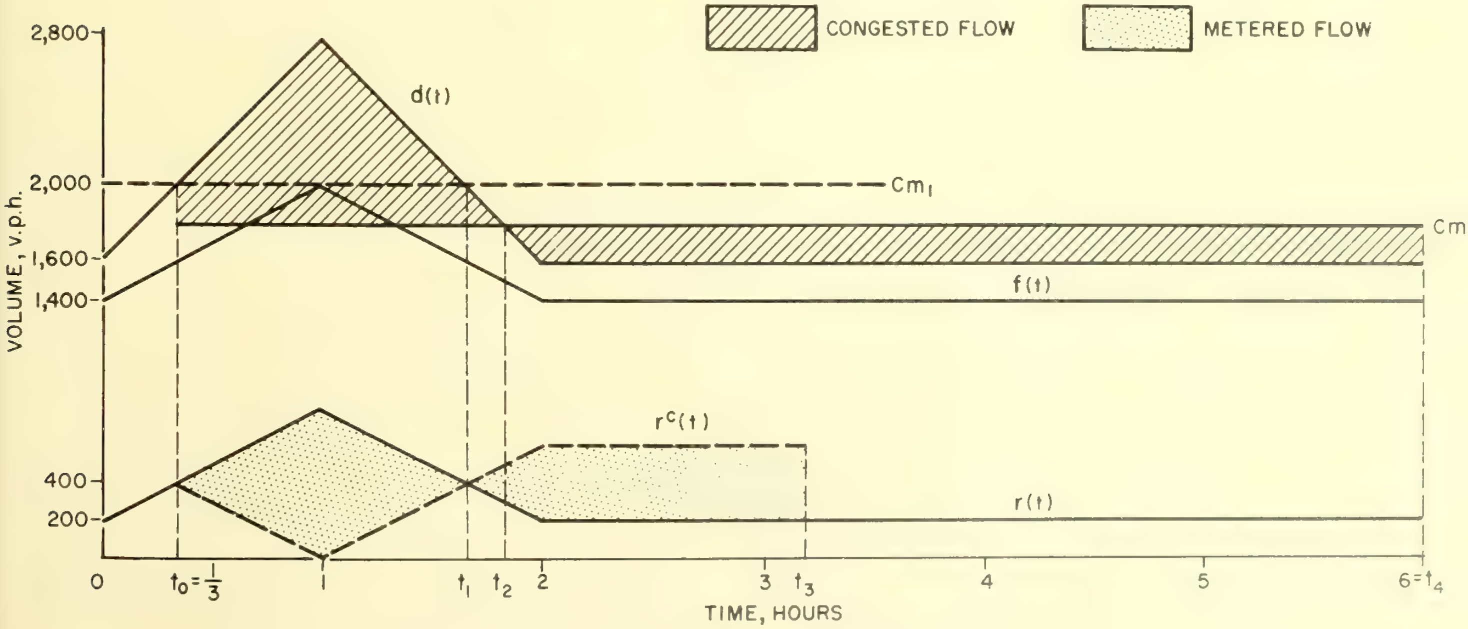

Figure 5.—Solution of ramp metering problem.

The calculated total delay is 799 vehicle-hours for the traffic flow situation where ramp metering is employed.

Calculation of length of congested period or ramp metering period

The length of the congested period is

$$t_4-t_0=6-\tfrac{1}{3}=5\tfrac{2}{3}\text{ hours.}$$

The length of the period of ramp metering is $t_3-t_0=3\tfrac{1}{6}-\tfrac{1}{3}=2\tfrac{5}{6}$ hours.

Analysis of Economy of Sample Metering Scheme

The improvement conceived for the previous example of the benefits to be derived from volume controls was an isolated ramp metering location, and the formulated ramp metering scheme provided for penalizing the on-ramp flow to benefit the freeway flow. The analysis quantified the benefits to be derived from this type of installation in terms of vehicle-hours saved. The control philosophy used in the example was to allow the freeway to operate in an uncongested state at capacity, which means that the freeway was assumed to be operating at 2,000 vehicles per hour and that the average speeds would be in the range of 35 to 40 miles per hour.

As mentioned, the ramp traffic bore the full burden of the delay; however, this resulted in a substantial reduction in total delay, the capability for maintaining capacity flow, and higher average speeds on the freeway. It would be possible to set the metering rate to maintain a higher level of service on the freeway, but the benefits in terms of total delay would not be as evident.

Obviously there are no single-lane freeways, but an analysis for a single-lane situation actually errs on the conservative side. An analysis for a multilane highway under conditions in which the merge point capacity was exceeded would show even greater benefits from the application of this particular method.

For the example illustrated there are only two logical alternatives: One is to do nothing, and the other is to install equipment to implement the control procedures described. Peak period volume controls are not usually affected by the average daily traffic (ADT); highways that require metering installations or other volume controls are those on which demand regularly exceeds the capacity of the facility. It is not necessary to forecast growth rate, because there will be little opportunity for growth. In the example, the freeway operated at 1,600 vehicles per hour in the off-peak period (freeway and ramp flow combined). Increasing demand would require longer periods of control.

There are only three surveillance facilities in the country—those at Detroit, Chicago, and Houston—that have had appreciable experience in surveillance and control techniques. Although exact costs of the initial investment and of operations are not readily available for these projects, approximate costs were obtained to provide an accurate estimate. As a single facility is being dealt with, the cost of the total freeway is not needed; only the cost of the metering installation, the central control center, and related operating expenses have to be estimated. Similarly, only the differences in the road user costs caused by congestion need be considered.

It was assumed that the useful life of the equipment is 15 years and that there would be no salvage value. Three vestcharge rates of 8, 10, and 12 percent were selected for use in the calculations. The cost of accidents was not considered in the analysis, and consequently, the benefits to be derived from a reduction in accidents were not determined. It was decided that the benefit-cost-ratio method would be used rather than the rate-of-return method. Computation of the road user cost required an estimate of the number of speed change cycles during the period of congestion, or metering period. This was computed by assuming conditions: the rate of acceleration and deceleration for a speed change cycle is 5.5 feet per second [2] (a speed change cycle is defined by Winfrey (*6*) as "reducing speed from and returning to an initial speed"); the speed before entering the congested section and after leaving it is 40 m.p.h.; traffic flow is stop and go i the congested area with the vehicle repeatedl accelerating to 35 m.p.h. and then decelerating to zero. Under these conditions, only th major and more prominent stoppages in th traffic flow were considered.

Using these assumptions and the compute total delay it is possible to compute the numbe of speed change cycles at 35 m.p.h. The number of speed change cycles at 40 m.p.h. is equa to the number of vehicles through the sectio in the period of congestion or ramp metering It was also assumed that there were no commercial vehicles in the traffic stream. The results of these computations above are show in table 2; the costs for the speed change cycles shown are taken from tables derived by Winfrey (*6*).

The vehicle-running costs shown in table 2 and the costs for delay in vehicle-hours shown in table 3 are daily costs. As the cost of time is a moot subject, the total costs for time were calculated at three different dollar amounts—$1.00, $1.55, and $7.20 per hour per person. The differences in costs between the uncontrolled merge and controlled merge were calculated and are shown in table 4. The costs of equipment and operations incurred on the different surveillance projects are shown in

Table 2.—Calculation of speed-change cycles and vehicle-running costs incurred

Type of operation	Vehicle delay per day	Time used per cycle per vehicle	Speed-change cycles	Cost	
				Per 1,000 cycles	Per day
	Vehicle hours	*Seconds*	*Number*		
Controlled:					
From 40 m.p.h	7	23, 34	1, 133	$15. 76	$17. 86
From 35 m.p.h	792	18. 64	152, 961	12. 24	1, 872. 24
Uncontrolled:					
From 40 m.p.h	64	21. 34	10, 800	15. 76	170. 21
From 35 m.p.h	2, 360	18. 64	455, 794	12. 24	5, 578. 92

Table 3.—Cost of time delay caused by congestion or metering

Type of operation	Total time delay		Total cost		
	Vehicle delay	Person delay	@ $1.00 per hour	@ $1.55 per hour	@ $7.20 per hour
	Hours	*Hours*			
Controlled	799	1, 200	$1, 200	$1, 860	$8, 640
Uncontrolled	2, 404	3, 606	3, 606	5, 589	25, 963

Table 4.—Differences in road-user costs between uncontrolled merging and controlled merging

Classification	Cost differences		
	Per day	Per year	Cumulative annual totals [1]
Cost of speed changes:			
From 40 m.p.h	$152. 35	$39, 611	------------
From 35 m.p.h	3, 706. 68	963, 737	------------
TOTAL	------------	1, 003, 348	------------
Cost of time:			
@ $1.00 per person-hour	2, 406. 00	625, 560	$1, 628, 908
@ $1.55 per person-hour	3, 729. 00	969, 540	1, 972, 888
@ $7.20 per person-hour	17, 323. 00	4, 505, 980	5, 509, 328

[1] Totals obtained by adding annual time costs to annual total speed-change costs ($1,003,348).

able 5. It was decided that the Detroit project osts would be used to approximate the costs f the hypothetical ramp-metering installa- ion considered here; actually a fixed-time etering installation could be installed and aintained at a much lower annual cost. Table shows the benefit-cost ratios calculated for he various factors. It can be seen that the amp-metering installation can be easily justi- ed according to the benefit-cost analysis.

Summary

Several methods of ramp metering are urrently being studied. Ramp closure and xed time metering are relatively inflexible ethods of control; however, these methods ave proved to be useful in eliminating re- urring congestion. Dynamic ramp metering ncluding gap-availability metering and system demand-capacity metering would seem to require more definitive analytic descriptions of the traffic flow interactions than are available; current research in dynamic ramp metering indicates that ramp metering may eventually contribute in a major way to the elimination of freeway congestion. Metering systems have been developed to the extent, that congestion has been significantly reduced but not completely eliminated. In developing metering systems care should be taken to ensure that the storage requirements on ramps and frontage roads do not cause detrimental traffic conditions to develop in the flow of traffic on adjacent streets and highways. Ramp-metering techniques have been proved effective in decreasing travel time throughout the system, increasing volumes and average speeds and reducing accidents.

Economic constraints could prove to be effective in regulating demand; however, the effect of economic parameters on traffic flow has not been determined, and at present there is no way of predicting exactly how the imposition of toll constraints would affect traffic flow. It does appear that institution of the toll or high parking-fee type of constraint could adversely affect further economic development of those areas incorporating them.

Dynamic diversion of traffic consists of monitoring traffic conditions on the freeway networks and informing the driver of the alternate routes that he should use to prevent congestion in individual links of the network and to provide optimum and maximum utilization of the system.

The use of staggered hours as the sole means of regulating demand is not feasible in most instances, but there are some circum-

Table 5.—Annual costs of operating different surveillance projects

Facility or project	Initial cost	Vestcharge rate [1]	Capital recovery factor (15 years)	Annual cost
John Lodge Expressway, Detroit, Mich. (National Proving Ground):		*Percent*		
TV cameras and monitors	$140,000	8	0.11683	$16,356
		10	0.13147	18,406
		12	0.14682	20,555
Control cable [2]				22,500
Detectors and traffic-control equipment	310,525	8	0.11683	36,279
		10	0.13147	40,825
		12	0.14682	45,491
Computer [3]				36,000
Annual-power costs				9,700
Annual equipment maintenance costs				14,300
Personnel [4]				67,500
TOTAL @ 8-percent rate				202,635
TOTAL @ 10-percent rate				209,231
TOTAL @ 12-percent rate				216,046
Gulf Freeway, Houston, Tex.:				
Control center [3]				9,600
Television system [3]				108,000
Ramp-control equipment				60,000
Office costs				3,000
Personnel				30,000
Detector system [3]				23,000
TOTAL				233,600
Chicago area expressway [4]				250,000

[1] The cost of money (investment charge) for which a noncash benefit is realized, as in the public-works field. Term coined by Robley Winfrey, Group Leader, Engineering Economics Group, Bureau of Public Roads.
[2] Equipment leased.
[3] Equipment rented.
[4] Cost estimated.

Table 6.—Summary of economic analysis of a ramp-metering installation

Vest-Charge rate [1]	Time costs excluded			Time costs included								
				@ $1.00 per hour			@ $1.55 per hour			@ $7.20 per hour		
	Annual benefits	Annual costs	Benefit-cost ratio	Annual benefits	Annual costs	Benefit-cost ratio	Annual benefits	Annual costs	Benefit-cost ratio	Annual benefits	Annual costs	Benefit-cost ratio
Percent												
8	$853,348	$52,635	16	$1,478,908	$52,635	28	$1,822,888	$52,635	34	$5,357,328	$52,635	101
10		59,231	14		59,231	25		59,231	30		59,231	90
12		66,046	12		66,046	22		66,046	27		66,046	81

[1] The cost of money (investment charge) for which a noncash benefit is realized, as in the public-works field. Term coined by Robley Winfrey, Group Leader, Engineering Economics Group; ureau of Public Roads.

stances in which the staggering of hours may be warranted.

Development of these methods and their use, either individually or in combination, promise to provide transportation engineers with effective tools for eliminating congestion.

A procedure was developed for calculating ramp-metering rates, length of queue, total delay, and the length of the period of congestion or ramp metering. This procedure was used to calculate the required metering rates and other factors for a simplified ramp-metering situation.

A benefit-cost analysis was performed to determine whether the ramp-metering alternative was a better choice than the uncontrolled-freeway alternative. The benefit-cost ratios developed range from 12 to 101.

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Ultraviolet Spectrophotometric Detection of Adulteration in Traffic Paint Vehicles

(*Continued from p. 257*)

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Travel by Motor Vehicles in 1965 and 1966

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

by THEODORE S. DICKERSON, JR., and W. JOHNSON PAGE,
Highway Research Engineers, Current Planning Division

MOTOR-VEHICLE travel in the United States in 1966 totaled 930.5 billion ehicle-miles, an increase of 4.8 percent over he travel in 1965. The travel data were ompiled by the Bureau of Public Roads from nformation supplied by the State highway epartments and toll authorities. Total travel or 1967, based on information for the first months of the year, is estimated at 967 illion vehicle-miles, a 3.9 percent increase ver 1966.

The term vehicle-miles and the other echnical terms used in this article are defined n the following statements:

Vehicle-miles.—Vehicle-miles refers to the mount of travel by one motor-vehicle raveling 1 mile and includes travel on all ighways and streets in the United States.

Trailer combinations.—A trailer combination s a truck or truck tractor pulling one or more railers and/or a semitrailer.

Motor-fuel comsumption.—Motor-fuel conumption is the total consumption of motor uel by highway vehicles for the year, obtained rom State records.

Motor-fuel consumption rate.—Motor-fuel onsumption rate is the average rate of motor uel usage in miles per gallon (m.p.g.).

Because of recent rapid increases in motorycle usage, separate estimates of travel for his vehicle type are shown for the first time. Motorcycle travel is estimated to have inreased 32.4 percent from 1965 to 1966. The stimates for motorcycles, which are less detailed and less reliable than for other vehicle ypes, are based on special counts obtained by he State highway departments beginning in 965 and on very limited data on usage haracteristics from industry and user sources.

The proportions of travel by road system nd vehicle type changed little from 1965 to 966. Of the 1966 travel, 35.2 percent was on nain rural roads comprising 14 percent of the Nation's total of 3.7 million miles of roads nd streets. Some 50.5 percent of the travel vas on urban streets, which comprise 14 percent of the total mileage. Travel on local rural roads was only 14.3 percent, although these roads are 72 percent of the total mileage.

Passenger cars represented 82 percent of the vehicles registered and did 80 percent of the travel in 1966; motorcycles, 2 percent of the vehicles and less than 1 percent of the travel; trucks and truck combinations, 16 percent of the vehicles and 19 percent of the travel. Similar figures for buses were less than 1 percent.

Average vehicle performance in 1966 differed very little from that reported for 1965. The average motor vehicle traveled 9,698 miles in 1966, half of it in cities, and consumed 778 gallons of fuel at a rate of 12.47 miles per gallon. The average passenger car traveled 9,506 miles and consumed 679 gallons of fuel at a rate of 14.00 miles per gallon. In 1966, the average truck and the average commercial bus traveled a little more than in 1965, but their average rates of fuel consumption did not change appreciably.

The average annual travel for single-unit trucks, however, decreased from 10,003 miles in 1965 to 9,588 miles in 1966. Since these trucks are used extensively in the construction industry, substantial changes in construction activity are likely to affect single-unit truck travel. New construction placed in the United States in 1966, in constant (1957–59) dollars, decreased slightly from 1965.

The travel and related information for 1966 and revised information for 1965 are shown in table 1 by road system and vehicle type. Such data have been reported in PUBLIC ROADS, *A Journal of Highway Research*, for a number of years; the latest for 1965 appeared in vol. 34, No. 6, February 1967, pp. 132 and 133.

The comparable State-by-State estimates of 1965 travel, shown in table 2, became available as a byproduct of the preparation of the report to the Congress, *1968 Estimate of the Cost of Completing the National System of Interstate and Defense Highways.* Each State highway department prepared an estimate of actual travel in 1965, as a foundation for traffic forecasts needed for this report.

According to the State estimates, the traveled way of the Interstate System carried 147.2 billion vehicle-miles, or 16.6 percent of the total 1965 travel on all roads and streets. The traveled way consisted of 18,000 miles of Interstate System highways now in use and 23,000 miles of existing connecting highways; service for this total mileage will be provided by the completed Interstate System. From the State estimates it is expected that by 1975 the 41,000 mile Interstate System, comprising little more than 1 percent of the total road and street mileage of the United States, will carry more than 20 percent of the total 1,213 billion miles of travel estimated for 1975.

According to the State estimates of 1965 travel, all Federal-aid systems combined, which includes about 25 percent of all roads and streets, carried 65 percent of all travel. Because of their principal use in a report to the Congress, the State estimates of 1965 were made according to a system classification and rural-urban distinction directly related to the Federal-aid program. In the Federal-aid law, an urban area is "an area including and adjacent to a municipality or other urban place having a population of 5,000 or more . . . ." In the annual estimates reported in table 1, however, urban signifies the areas within the political boundaries of municipalities such as cities, boroughs, and villages. As a consequence, urban travel in 1965 as shown in table 1 was 50.5 percent of the total, but by the State estimates it was 48.6 percent.

In recent years the annual travel estimates reported in table 1 have been developed in part from trend indicators, extending from a base of comprehensive studies by the States of travel in 1957. The revised estimate of total travel in 1965 is only 0.02 percent more than the original estimate. One of the larger differences was in the proportion of urban travel, which was revised from 48.0 percent of total travel to 50.6 percent.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1966 and revised for 1965 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main [2] rural roads	Local rural roads	All rural roads	Urban streets	Total			Total	Average per vehicle	
1966										
Personal passenger vehicles:	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles gallon*
Passenger cars [3]	....	....	....	....	744, 844	78, 353	9, 506	53, 220	679	14. 00
Motorcycles [3]	....	....	....	....	6, 896	1, 753	3, 930	92	52	75. 00
ALL PERSONAL PASSENGER VEHICLES	247, 626	103, 746	351, 372	400, 368	751, 740	80, 106	9, 384	53, 312	666	14. 10
Buses:										
Commercial	941	196	1, 137	1, 871	3, 008	84. 5	35, 598	637	7, 538	4. 72
School	712	798	1, 510	334	1, 844	238. 7	7, 725	259	1, 085	7. 12
ALL BUSES	1, 653	994	2, 647	2, 205	4, 852	323. 2	15, 012	896	2, 772	5. 42
ALL PASSENGER VEHICLES	249, 279	104, 740	354, 019	402, 573	756, 592	80, 429	9, 407	54, 208	674	13. 96
Cargo vehicles:										
Single-unit trucks	57, 143	26, 774	83, 917	56, 976	140, 893	14, 694	9, 588	13, 636	928	10. 33
Trailer combinations	21, 277	1, 507	22, 784	10, 228	33, 012	823	40, 112	6, 779	8, 237	4. 87
ALL TRUCKS	78, 420	28, 281	106, 701	67, 204	173, 905	15, 517	11, 207	20, 415	1, 316	8. 52
ALL MOTOR VEHICLES	327, 699	133, 021	460, 720	469, 777	930, 497	95, 946	9, 698	74, 623	778	12. 47
1965 REVISED										
Personal passenger vehicles:										
Passenger cars	....	....	....	....	706, 386	75, 252	9, 387	50, 206	667	14. 07
Motorcycles	....	....	....	....	5, 208	1, 382	3, 770	69	50	75. 00
ALL PERSONAL PASSENGER VEHICLES	236, 777	96, 635	333, 412	378, 182	711, 594	76, 634	9, 286	50, 275	656	14. 15
Buses:										
Commercial	932	194	1, 126	1, 893	3, 019	85. 0	35, 518	645	7, 588	4. 68
School	687	758	1, 445	318	1, 763	229. 3	7, 689	249	1, 086	7. 08
ALL BUSES	1, 619	952	2, 571	2, 211	4, 782	314. 3	15, 215	894	2, 844	5. 35
ALL PASSENGER VEHICLES	238, 396	97, 587	335, 983	380, 393	716, 376	76, 948	9, 310	51, 169	665	14. 00
Cargo vehicles:										
Single-unit trucks	52, 771	28, 177	80, 948	59, 169	140, 117	14, 008	10, 003	13, 504	964	10. 38
Trailer combinations	20, 459	1, 382	21, 841	9, 478	31, 319	787	39, 795	6, 431	8, 172	4. 87
ALL TRUCKS	73, 230	29, 559	102, 789	68, 647	171, 436	14, 795	11, 587	19, 935	1, 347	8. 60
ALL MOTOR VEHICLES	311, 626	127, 146	438, 772	449, 040	887, 812	91, 743	9, 677	71, 104	775	12. 49

[1] For the 50 States and District of Columbia.

[2] Main rural roads include roads on the State highway systems, roads of the Interstate System, other mileage on Federal-aid systems, and major toll roads, which approximate in 1965 523,000 road miles and in 1966, 527,000 road miles.

[3] Separate estimates of passenger car and motorcycle travel are not available by highway category.

Table 2.—Vehicle-miles of travel in 1965 on all roads and streets, free and toll, by State and highway system, estimated by State highway departments in 1967

(Millions)

Division	State	Federal-aid highway system																	Not on Federal-aid systems				Sub-total rural	Sub-total urban and municipal	Total
		Interstate rural			Interstate urban			Sub-total Inter-state	Other Primary			Secondary					Total Federal-aid rural	Total Federal-aid urban	Total Federal-aid	Other State rural 09	Other State munici-pal 10	Local rural 11	Local munici-pal 12			
		Final 01	Traveled 1/ 31	Total rural	Final 02	Traveled 1/ 32	Total urban		Rural 03	Urban 04	Total	State rural 05	State urban 06	Local rural 07	Local urban 08	Total										
New England	Connecticut	622	163	785	1,637	414	2,051	2,836	1,193	1,438	2,631	899	668	7	67	1,641	2,884	4,224	7,108	428	2,537	161	2,766	3,473	9,527	13,000
	Maine	413	129	542	53	71	124	666	1,243	420	1,663	792	121	-	-	913	2,577	665	3,242	743	284	238	339	3,558	1,287	4,845
	Massachusetts	951	154	1,105	996	1,071	2,067	3,172	2,384	4,273	6,657	529	475	1,067	1,463	3,534	5,085	8,278	13,363	158	1,208	835	5,737	6,078	15,223	21,301
	New Hampshire	322	52	374	54	48	102	476	952	320	1,272	680	81	4	3	768	2,010	506	2,516	237	59	262	401	2,509	966	3,475
	Rhode Island	13	85	98	308	154	462	560	488	757	1,245	172	171	22	177	542	780	1,567	2,347	70	146	18	1,236	868	2,949	3,817
	Vermont	131	192	323	23	59	82	405	643	119	762	259	1	136	16	412	1,361	218	1,579	7	*	188	198	1,556	416	1,972
	Total	2,452	775	3,227	3,071	1,817	4,888	8,115	6,903	7,327	14,230	3,331	1,517	1,236	1,726	7,810	14,697	15,458	30,155	1,643	4,234	1,702	10,676	18,042	30,369	48,410
Middle Atlantic	New Jersey	197	801	998	925	1,725	2,650	3,648	2,868	4,555	7,423	58	47	1,786	1,588	3,579	5,710	8,940	14,650	1,043	1,968	4,088	11,812	10,841	22,720	33,561
	New York	2,230	276	2,506	4,397	922	5,319	7,825	8,084	10,633	18,717	1,686	1,091	2,688	1,364	6,829	14,964	18,407	33,371	41	57	7,983	14,897	22,988	33,361	56,349
	Pennsylvania	3,167	1,361	4,528	1,828	964	2,792	7,320	7,571	5,258	12,829	5,300	3,081	46	90	8,517	17,445	11,221	28,666	3,062	3,785	3,912	9,840	24,419	24,846	49,265
	Total	5,594	2,438	8,032	7,150	3,611	10,761	18,793	18,523	20,446	38,969	7,044	4,219	4,520	3,142	18,925	38,119	38,568	76,687	4,146	5,810	15,983	36,549	58,248	80,927	139,175
South Atlantic (North)	Delaware	43	-	43	124	159	282	325	938	607	1,545	296	192	-	-	488	1,277	1,081	2,358	-	-	80	62	1,357	1,143	2,500
	Dist. of Col. 2/	-	-	-	134	167	301	301	-	980	980	-	-	-	486	486	-	1,767	1,767	-	-	-	714	-	2,481	2,481
	Maryland	544	332	876	1,043	968	2,011	2,887	2,600	2,030	4,630	1,428	763	400	376	2,967	5,304	5,180	10,484	567	68	2,890	2,265	8,761	7,513	16,274
	Virginia	1,997	1,568	3,565	338	887	1,225	4,790	4,695	2,021	6,716	2,331	416	1,723	367	4,837	12,314	4,029	16,343	69	322	1,450	2,538	13,833	6,889	20,722
	West Virginia	242	514	756	50	247	297	1,053	1,827	627	2,454	1,373	103	825	29	2,330	4,781	1,056	5,837	4	18	277	1,050	5,062	2,124	7,186
	Total	2,826	2,414	5,240	1,689	2,427	4,116	9,356	10,060	6,265	16,325	5,428	1,474	2,948	1,258	11,108	23,676	13,113	36,789	640	408	4,697	6,629	29,013	20,150	49,163
South Atlantic (South)	Florida	1,181	1,182	2,363	860	1,068	1,928	4,291	4,592	2,745	7,337	4,179	1,905	406	126	6,616	11,540	6,704	18,244	1,071	937	1,857	6,483	14,468	14,124	28,592
	Georgia	988	1,267	2,255	943	574	1,517	3,772	5,056	1,481	6,537	2,269	457	1,157	327	4,210	10,737	3,782	14,519	134	352	1,654	4,921	12,525	9,055	21,580
	North Carolina	1,036	1,091	2,127	292	231	523	2,650	3,929	1,352	5,281	4,371	727	3,603	867	9,568	14,030	3,469	17,499	64	236	1,338	2,724	15,432	6,429	21,861
	South Carolina	738	694	1,432	62	141	203	1,635	3,852	1,234	5,086	2,670	388	114	4	3,176	8,068	1,829	9,897	264	784	319	318	8,651	2,931	11,582
	Total	3,943	4,234	8,177	2,157	2,014	4,171	12,348	17,429	6,812	24,241	13,489	3,477	5,280	1,324	23,570	44,375	15,784	60,159	1,533	2,309	5,168	14,446	51,076	32,539	83,615
East North Central	Illinois	1,951	1,450	3,401	3,656	882	4,538	7,939	7,852	6,504	14,356	1,024	468	2,377	196	4,065	14,654	11,706	26,360	1,370	3,332	2,666	11,821	18,690	26,859	45,549
	Indiana	1,414	1,532	2,946	238	1,238	1,476	4,422	5,758	2,516	8,274	2,748	570	1,723	621	5,662	13,175	5,183	18,358	131	214	1,031	5,805	14,337	11,202	25,539
	Michigan	2,423	266	2,689	2,006	1,722	3,728	6,417	6,643	4,735	11,378	1,333	214	6,173	781	8,501	16,838	9,458	26,296	27	57	4,033	11,149	20,898	20,664	41,562
	Ohio	3,414	673	4,087	2,299	1,515	3,814	7,901	7,618	4,929	12,547	4,180	1,387	2,190	1,859	9,616	18,075	11,989	30,064	104	401	4,691	11,490	22,870	23,880	46,750
	Wisconsin	1,044	268	1,312	235	253	488	1,800	5,030	2,030	7,060	1,560	463	1,425	833	4,281	9,327	3,814	13,141	36	39	1,096	4,863	10,459	8,716	19,175
	Total	10,246	4,189	14,435	8,434	5,610	14,044	28,479	32,901	20,714	53,615	10,845	3,102	13,888	4,290	32,125	72,069	42,150	114,219	1,668	4,043	13,517	45,128	87,254	91,321	178,575
West North Central	Iowa	681	381	1,062	159	67	226	1,288	5,025	1,361	6,386	-	-	1,455	285	1,740	7,542	1,872	9,414	77	47	841	2,354	8,460	4,273	12,733
	Kansas	605	238	843	280	111	391	1,234	3,606	993	4,599	543	20	1,372	310	2,245	6,364	1,714	8,078	77	47	879	3,349	7,320	5,110	12,430
	Minnesota	194	724	918	512	830	1,342	2,260	4,741	2,000	6,741	1,013	32	2,411	145	3,601	9,083	3,519	12,602	3	115	1,253	3,207	10,339	6,841	17,180
	Missouri	1,252	1,006	2,258	1,158	568	1,726	3,984	4,961	1,849	6,810	2,249	376	20	34	2,679	9,488	3,985	13,473	158	488	1,850	5,894	11,496	10,367	21,863
	Nebraska	412	296	708	39	36	75	783	2,579	578	3,157	587	15	611	63	1,276	4,485	731	5,216	2	-	975	1,757	5,462	2,488	7,950
	North Dakota	255	171	426	18	49	67	493	1,188	113	1,301	181	3	429	14	627	2,224	197	2,421	1	2	641	374	2,866	573	3,439
	South Dakota	293	302	595	17	14	31	626	1,538	186	1,724	213	21	404	24	662	2,750	262	3,012	20	7	615	329	3,385	598	3,983
	Total	3,692	3,118	6,810	2,183	1,675	3,858	10,668	23,638	7,080	30,718	4,786	467	6,702	875	12,830	41,936	12,280	54,216	338	706	7,054	17,264	49,328	30,250	79,578
East South Central	Alabama	381	1,122	1,503	58	740	798	2,301	3,476	1,459	4,935	1,249	216	1,603	198	3,266	7,831	2,671	10,502	385	102	633	2,702	8,849	5,475	14,324
	Kentucky	824	849	1,673	478	394	872	2,545	3,326	1,075	4,401	2,939	381	410	90	3,820	8,348	2,418	10,766	813	510	970	1,695	10,131	4,623	14,754
	Mississippi	351	564	915	87	437	524	1,439	3,189	578	3,767	712	77	865	158	1,812	5,681	1,337	7,018	8	19	600	1,219	6,289	2,575	8,864
	Tennessee	348	1,557	1,905	419	837	1,256	3,161	4,052	1,840	5,892	967	222	625	38	1,852	7,549	3,356	10,905	53	5	1,665	3,417	9,267	6,778	16,045
	Total	1,904	4,092	5,996	1,042	2,408	3,450	9,446	14,043	4,952	18,995	5,867	896	3,503	484	10,750	29,409	9,782	39,191	1,259	636	3,868	9,033	34,536	19,451	53,987
West South Central	Arkansas	376	543	919	238	89	327	1,246	2,489	795	3,284	1,719	230	276	26	2,251	5,403	1,378	6,781	33	44	664	1,262	6,100	2,684	8,784
	Louisiana	393	1,104	1,497	239	562	801	2,298	2,367	1,128	3,495	3,069	482	11	1	3,563	6,944	2,412	9,356	408	450	610	2,130	7,962	4,992	12,954
	Oklahoma	954	264	1,218	431	414	845	2,063	4,318	1,221	5,539	916	193	676	207	1,992	7,128	2,466	9,594	197	91	964	3,042	8,289	5,599	13,888
	Texas	1,763	2,588	4,351	3,782	2,267	6,049	10,400	10,201	6,402	16,603	6,963	2,482	-	-	9,445	21,515	14,933	36,448	1,160	719	2,460	12,212	25,135	27,864	52,999
	Total	3,486	4,499	7,985	4,690	3,332	8,022	16,007	19,375	9,546	28,921	12,667	3,387	963	234	17,251	40,990	21,189	62,179	1,798	1,304	4,698	18,646	47,486	41,139	88,625
Mountain	Arizona	612	1,492	2,104	239	330	569	2,673	1,064	109	1,173	524	43	538	1,590	2,695	4,230	2,311	6,541	156	20	326	1,254	4,712	3,585	8,297
	Colorado	613	615	1,228	363	373	736	1,964	1,914	1,381	3,295	796	119	-	-	915	3,938	2,236	6,174	48	98	1,244	2,459	5,230	4,793	10,023
	Idaho	312	334	646	16	75	91	737	1,167	129	1,296	304	22	263	36	625	2,380	278	2,658	4	7	627	486	3,011	771	3,782
	Montana	200	556	756	8	69	77	833	1,220	178	1,398	366	48	-	-	414	2,342	303	2,645	-	-	665	390	3,007	693	3,700
	Nevada	395	314	709	-	143	143	852	624	183	807	244	59	71	163	537	1,648	548	2,196	24	20	90	490	1,762	1,058	2,820
	New Mexico	644	659	1,303	67	225	292	1,595	1,399	479	1,878	746	124	6	-	876	3,454	895	4,349	82	412	349	613	3,885	1,920	5,805
	Utah	136	913	1,049	134	359	493	1,542	736	390	1,126	499	408	92	122	1,121	2,376	1,413	3,789	33	117	375	582	2,784	2,112	4,896
	Wyoming	359	377	736	16	21	37	773	815	111	926	211	1	12	13	237	1,774	162	1,936	3	3	258	365	2,035	530	2,565
	Total	3,271	5,260	8,531	843	1,595	2,438	10,969	8,939	2,960	11,899	3,690	824	982	1,924	7,420	22,142	8,146	30,288	350	677	3,934	6,639	26,426	15,462	41,888
Pacific	California	1,597	5,109	6,706	7,663	4,293	11,956	18,662	9,500	12,916	22,416	2,020	976	3,902	3,328	10,226	22,128	29,176	51,304	531	1,296	13,443	29,210	36,102	59,682	95,784
	Oregon	1,030	281	1,311	486	110	596	1,907	2,069	1,224	3,293	994	260	647	393	2,294	5,021	2,473	7,494	3	12	1,210	1,796	6,234	4,281	10,515
	Washington	269	1,008	1,277	919	472	1,391	2,668	1,818	982	2,800	829	404	1,645	335	3,213	5,569	3,112	8,681	16	187	3,374	3,070	8,959	6,369	15,328
	Total	2,896	6,398	9,294	9,068	4,875	13,943	23,237	13,387	15,122	28,509	3,843	1,640	6,194	4,056	15,733	32,718	34,761	67,479	550	1,495	18,027	34,076	51,295	70,332	121,627
Total - All Divisions		40,310	37,417	77,727	40,327	29,304	69,691	147,418	165,198	101,224	266,422	70,990	21,003	46,216	19,313	157,522	360,131	211,231	571,362	13,925	21,622	78,648	199,086	452,704	431,939	884,643
	Alaska	-	-	-	-	-	-	-	267	96	363	78	30	-	-	108	345	126	471	-	-	100	204	445	330	775
	Hawaii	-	124	124	17	267	284	408	420	261	681	195	37	-	-	232	739	582	1,321	-	-	371	702	1,110	1,284	2,394
United States Total		40,310	37,541	77,851	40,344	29,571	69,975	147,826	165,885	101,581	267,466	71,263	21,070	46,216	19,313	157,862	361,215	211,939	573,154	13,925	21,622	79,119	199,992	454,259	433,553	887,812

1/ Traveled-way includes travel on highways in the Interstate System status of completion group 2. This group consists of mileage which is adequate for present traffic but will need further construction and improvement to bring it to full Interstate standards. In some instances these are divided highways with full control of access and in others they are two, three or four-lane undivided highways with no access control.

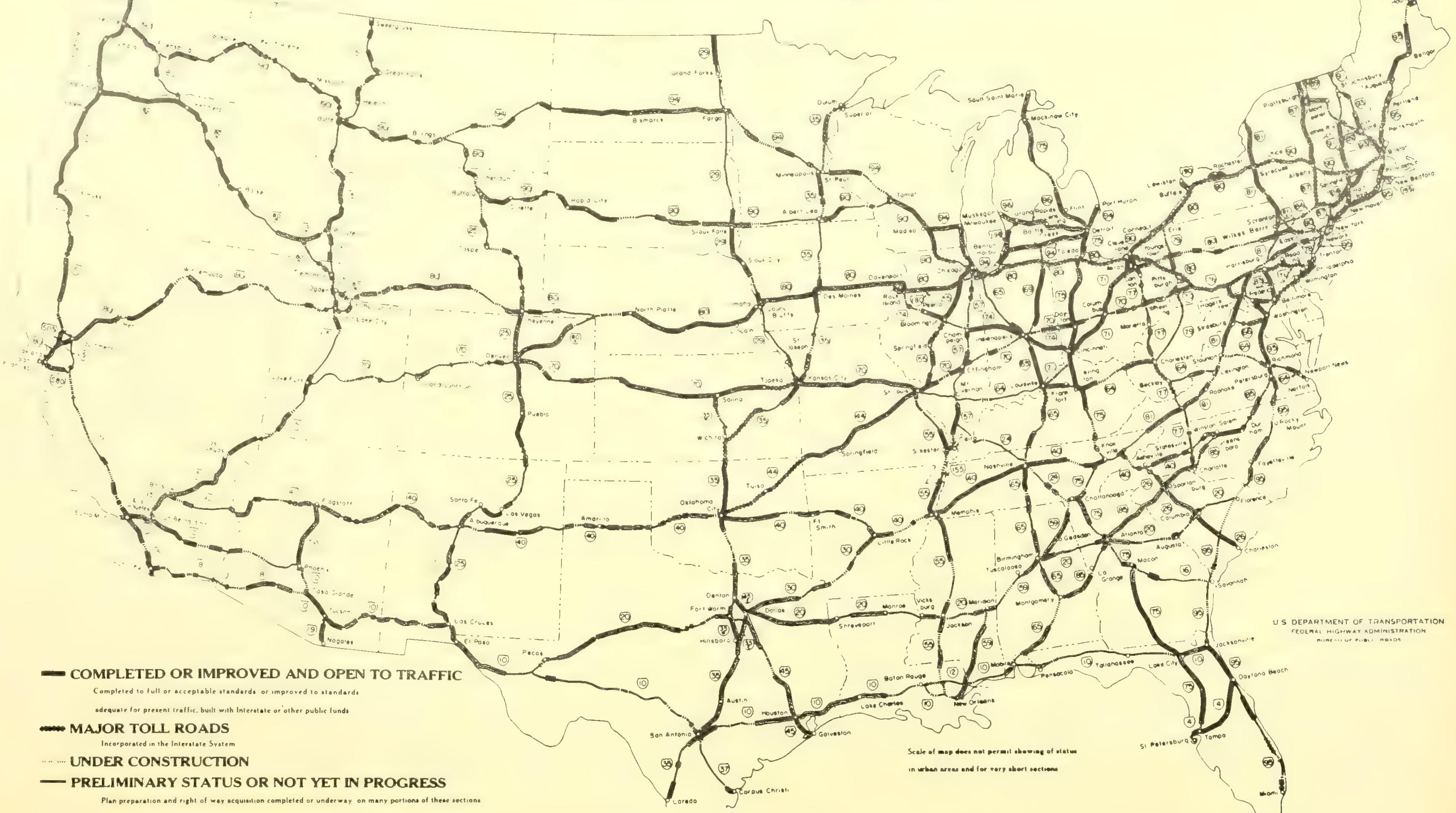
2/ Total vehicle-mile estimate provided by the District of Columbia Highway Department. Distribution of travel by highway system was estimated by the Planning Services Branch, Bureau of Public Roads.

3/ Bureau of Public Roads estimate based on data furnished by the Alaska Highway Department. Alaska has no interstate mileage and therefore did not prepare an estimate.

* Less than 0.5 million vehicle-miles.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

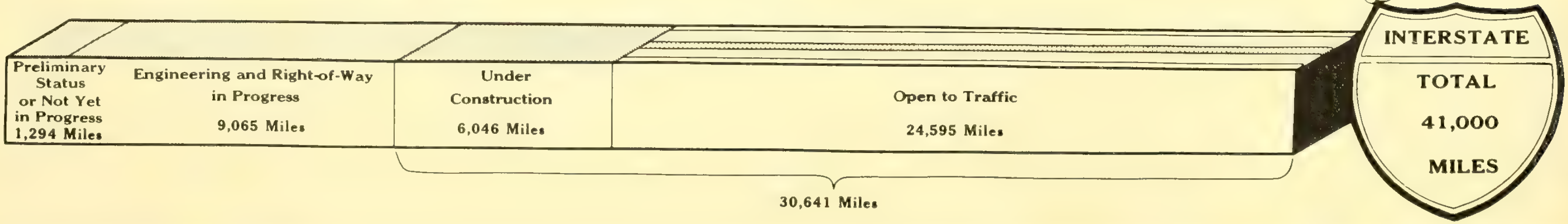
STATUS OF IMPROVEMENT AS OF SEPTEMBER 30, 1967



- COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards or improved to standards adequate for present traffic, built with Interstate or other public funds
- MAJOR TOLL ROADS
Incorporated in the Interstate System
- UNDER CONSTRUCTION
- PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right of way acquisition completed or underway on many portions of these sections

Scale of map does not permit showing of status in urban areas and for very short sections

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS



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April 1968

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

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Harry C. Secrest, *Managing Editor* ● Fran Faulkner, *Editor*

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ALAN S. BOYD, Secretary

FEDERAL HIGHWAY ADMINISTRATION
LOWELL K. BRIDWELL, Administrator

BUREAU OF PUBLIC ROADS
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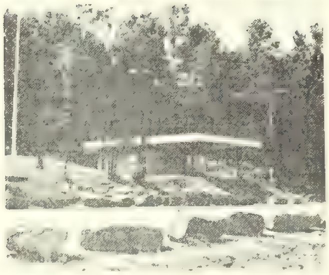

COVER

Roadside park on U.S. 167 near Salado Creek, south of Batesville, Arkansas.

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Benefits of Highway Beautification

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

Reported by FLOYD I. THIEL and JOHN YASNOWSKY, JR., Economists,[1] Economics and Requirements Division

The Congress expected the overall benefits of the Highway Beautification Act of 1965 to exceed the costs; but it requested additional information on the benefits and on the costs of carrying out the provisions of the act to provide a basis for future decisions on the beautification program.

Much information has been gathered on the direct financial costs of the program, and much has been learned about indirect costs in terms of the impact on the individuals and groups who expected such adverse effects from the act as short run losses that could be encountered by some outdoor advertising companies and roadside businesses. The benefits of the act have received little attention, perhaps partly because they are largely intangible and difficult to evaluate and measure. Despite this difficulty, decisionmakers must have some knowledge of the benefits and the costs if they are to make meaningful decisions.

In this article the authors emphasize the benefits of the beautification program and, in doing so, give little attention to the costs and the adverse effects of the act. However, there is no intent to imply that losses will not be incurred by some groups. The primary purpose of this article is to fill a gap that has existed concerning the nature and extent of the benefits to be expected from the beautification program.

Introduction

CONGRESSIONAL DECISIONS to undertake such programs as highway beautification require information about the benefits and costs of these programs. This need exists whether the information is provided explicitly in the form of a benefit-cost analysis or in some such form as a study of costs or of economic impact.

The Congress obviously felt that the benefits of the Highway Beautification Act of 1965 would outweigh the costs. In passing the act, the Congress recognized a need to learn more about the costs, the economic impact, and the public and private benefits of the act. To obtain information on these matters, the Congress requested a study of the beautification program.

Learning about the costs of the beautification program has presented no insurmountable problems; accordingly, at this point, information about the costs, including indirect costs, appears to be adequate. Information about the direct costs of the program, and of alternate programs, has been provided in a fairly comprehensive study conducted cooperatively by the State highway departments and the Bureau of Public Roads. Additional information about estimated costs has been provided by groups affected by the act, particularly members of the outdoor advertising industry.

The benefits of the highway beautification program can be estimated, at least in a general way, but these benefits cannot be quantified or described with the precision possible in dealing with money. In the study of the economic impact of the act (*1*),[2] which the Bureau of Public Roads conducted in cooperation with several State highway departments, the difficult problem of measuring benefits of highway beautification is recognized. In spite of the problems, some effort to evaluate the benefits must be made if meaningful decisions are to emerge.

Benefits of highway beautification were described not only in the Bureau's beautification impact study, but also in many of the 25 studies of the economic impact of the Highway Beautification Act conducted in many States throughout the country. These studies are listed at the end of this article. The need for information about benefits was underscored by a Congressional request at the 1967 House hearings on highway beautification. The discussion in this article, drawn from the Public Roads report of the economic impact study, is based partly on the attention that the Congress has focused on the need for information about benefits. The benefits referred to for one group, such as motorists, may be costs or disadvantages for others, such as outdoor advertisers. Some of the benefits referred to result from adjustments by those affected by the program rather than from beautification itself, and most of the benefits will occur only when the act becomes operational. Though these benefits are intangible and largely unmeasurable, they can be conveniently described when classified as: (1) General or overall benefits, (2) benefits from scenic enhancement, (3) benefits from outdoor advertising control, (4) benefits from junkyard control, and (5) benefits perceived by considering costs on a unit basis.

General or Overall Benefits

The overall benefits of highway beautification go primarily to people using the highways. The benefits anticipated include increased comfort, pleasure, convenience, and safety while traveling. Additional general or overall benefits that go beyond those resulting directly from scenic enhancement, billboard control, and junkyard control include: (1) An increased consciousness of the need for preserving the Nation's scenic resources, (2) the beneficial effect the highway beautification program will have on national and local economies, especially on local economies, and (3) the benefits resulting from the permanence of most highway beautification measures.

Increased interest in scenic resources

The current interest in the quality of the environment in the United States is unmistakable. Increasingly, the problems in this country are concerned with the quality of living rather than the amount of resources. "More and more people are concerned with water pollution, air pollution * * * solid wastes, the preservation of areas for outdoor recreation and for open space, the design and arrangement of both the urban and rural landscape. * * * Living in harmony with nature has become a matter of conscious attention and national policy. * * *" (*2*). Clearly, "there are strong indications of an awakening public appreciation of the amenities and acceptance of responsibility for preservation of our vanishing resources, the Nation's landscape" (*3*).

The Highway Beautification Act of 1965 cannot be properly credited with creating the current interest in esthetic highways or in environmental quality generally, but it can be credited with intensifying this interest. The act is focusing attention on "an often neglected aspect of highways".[3] News stories have credited the highway beautification program with adding impetus to such developments as: (1) Oil company efforts to design their service stations to blend with the scenery, and (2) attempts to protect trees by barricades during construction. An increase in subscriptions to the magazine *Landscape Architecture* has even been associated with the highway beautification program.

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[1] Alice C. Randill, James M. Montgomery, Anthony T. Cluff, and Edward M. Nolan also contributed work on which this article was based.

[2] Italic numbers in parentheses indicate references listed on page 8.

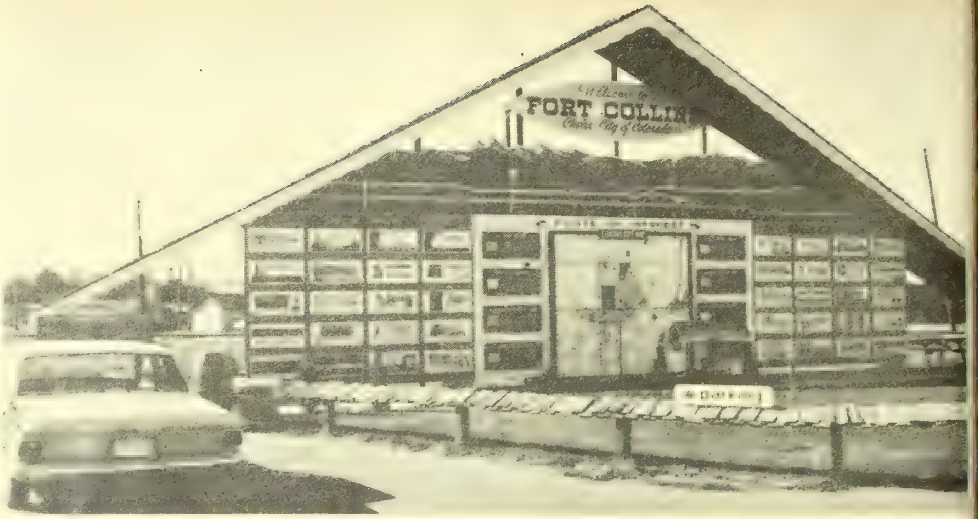
[3] From remarks by John T. Connor, former Secretary of Commerce, prepared for delivery before the Highway Transportation Congress, Washington Hilton Hotel Apr. 13 1966.

Effects on local economies

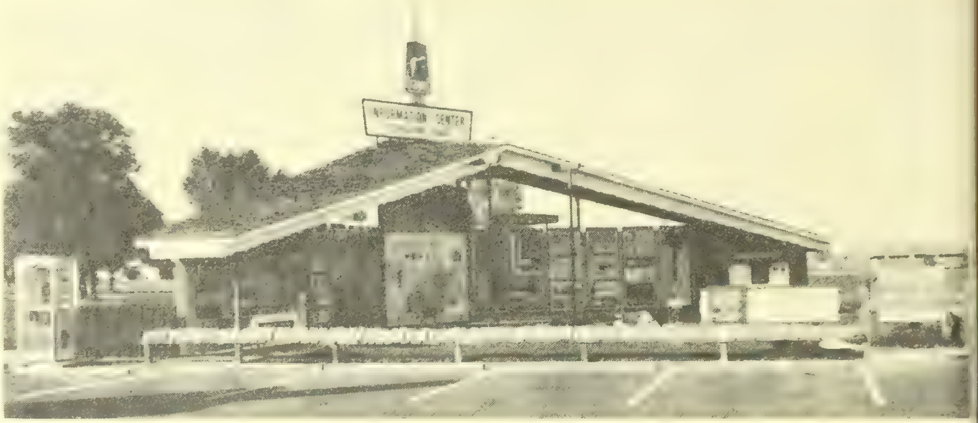
The economic stimulus provided by expenditures for highway beautification—that is, landscaping, screening, etc.—is fairly obvious. Although the effect on economic activity is generally less for highway expenditures than for other public expenditures where labor costs are more important, $1 spent for highway beautification will ordinarily result in total economic expenditures of more than $2 (*4*). This economic activity, though a benefit to some who are directly affected, results from the cost of the program. When the cost is borne by taxes, it will tend to offset the economic stimulant provided by the spending, at least on a national basis. Whether this economic activity produces a gain in national wealth or production depends on whether the taxing and spending for highway beautification result in the use of previously unused resources or in a more productive use of resources that were previously used.

It is not entirely clear whether the expenditures for scenic enhancement and junkyard control will stimulate economic activity sufficiently to offset the economic activity lost because of signboard removal. If, as is anticipated, an investment in landscaping or other comparable highway beautification projects encourages tourism, enhances land values, and eases driver tension, then the investment in highway beautification should yield economic gains comparable to those that were lost after removal of some billboards. This will be particularly true if, as is possible under the Highway Beautification Act, substitutes for commercial billboards, such as information centers and official right-of-way signs, are provided for the motorist.

Whether the gains will exceed the losses cannot be estimated with certainty at this time, but considerable insight into the matter has been provided by a study of the economy of Clinton County, Pa., in which an input-output model was used to provide forecasts such as the impact of highway construction and the effects of a strike at a local aircraft plant (*5*). The major finding from the study was the forecast that the county would experience a slight increase in economic activity as a result of the Highway Beautification Act. Apparently, counties like Clinton County that have no local outdoor advertising companies may experience less effect from the billboard control provisions than counties that have such companies. Of the 3,000 counties in the Nation, there may be a number with no outdoor advertising companies headquartered within their borders as the approximately 3,500 outdoor advertising companies tend to be grouped in counties with large cities.



On U.S. 287, Fort Collins, Colo.



On U.S. 34, Loveland, Colo.

Figure 1.—Examples of attractive and functional visitor orientation centers.

In terms of relative change, the impact in Clinton County will be slight. The total economic activity for the county as a whole will increase very little—less than a tenth of 1 percent. This would scarcely be felt in the economy, though there may be some dislocations evident owing to the income distribution within the community.

The beneficial effect that attractive surroundings have on economic activity is receiving more and more attention. It is now fairly common to hear such statements as "beauty is good business" (*6*) and "natural beauty and economic progress go together" (*7*). The burgeoning tourist industry has helped focus attention on the connection between pleasant surroundings and economic progress. Increasingly, communities are emphasizing their pleasant scenery in an effort to attract tourists. More and more, there appears to be agreement that cities need to "make a charming entrance" to "draw tourist dollars" (*8*). To help provide a pleasant entrance and to assist travelers, visitor information centers are being used in many locations. These are compatible with the highway beautification program, which provides for the use of information centers at safety rest areas (discussed later). Some typical tourist informatio centers are shown in figure 1.

Concern with the need for attracting tou ists has extended to travelers from abroad, an one of the potential benefits of highwa beautification is increased foreign touri trade. Under program budgeting concept highway money, about $4 million per yea has been earmarked for "Promotion of Na tional Prestige and International Trade" (*9* Recently a United States border improveme program was suggested to make United Stat entrances attractive and impressive. This ma have been in response to complaints tha border areas of the United States near Mexic are becoming shabby.

The increased opportunities that sceni highways may offer for business activity ar offset to some extent by losses in other area For this reason, highway beautificatio whether along the highway or on privat property bordering the highway, may no produce any substantial increase in revenu to the tourist industry on a national basis However, to the extent that foreign traveler can be encouraged to travel in America, o that Americans can be encouraged to trave more in this country, a gain in revenue i represented for the tourist industry. Such a gain still may not constitute a gain in national income if it is offset completely by a loss in other sectors of the economy. Ordinarily, where there are some unused resources, any gain to the tourist industry from domestic travel would be partly, but not completely, offset by losses in other sectors of the economy (*10*). Experience abroad may provide some insight into the relation between scenic highway and business experience. England and Switzerland with "more or less stringent systems" have been contrasted with Italy and Belgium, which Nairn rates "bad as America" with respect to roadside esthetics. Nairn suggests that the pleasing roadsides in England and Switzerland have been associated with business activity just as much as the roadsides in Belgium and Italy where roadside advertising is common.

Although most of the features of the Highway Beautification Act should encourage tourist travel and business related to highway travel, some affected businesses may not benefit. For example, safety rest areas tend to encourage highway-oriented business by making travel pleasant and safe. This should stimulate business. But rest areas also make it more convenient for motorists to picnic along the way, which may have an adverse effect on roadside restaurants.

The permanence of highway beautification

The benefits that highways, especially controlled-access highways, can yield as dividing lines or buffers between different land uses have been well established. Controlled-access highways are especially effective for this purpose, because of their permanence—their built-in resistance to obsolescence. Highway beautification enhances this feature of controlled-access highways.

Highways, like most other tangible objects, wear out over a period or become inadequate for the required service. Even highway right-of-way must be regarded as having a definite life, though right-of-way does ordinarily have a longer life than surfaces or structures. The fact that physical things will wear out and that benefits of the project will decline over the years obviously must be considered by decisionmakers when they evaluate the benefits and costs of investments, including Government investments. Because of this tendency for benefits to decline, it is ordinarily proper to base the evaluation of projects on economic life rather than on physical or useful life. Thus, even though right-of-way for a highway may have an unlimited physical life, it is better to consider the probable economic life (*11*, *12*); for example, the economic life for rural Interstate highways is often estimated to be 35 years for road surface, 65 years for structures, 80 years for grading, and 100 years for right-of-way.

With the exception of junkyard fencing and certain plantings that may not survive, most highway beautification measures should be quite enduring, possibly as enduring as the right-of-way. Some of the benefits of highway beautification may extend even beyond the economic life of the right-of-way as, for example, when the beautification occurs in areas where at least some of the benefits, such as the parklike effect of a well-landscaped open space, continue even after the highway is abandoned.

Although well-designed and well-located highways may have longer duration than the buildings or other manmade structures nearby, it is possible to surround highways with an environment that will be long lasting. The early parkways did this. In fact, the landscaping along some of these parkways has outlasted the roadway itself. The potential benefit of highway surroundings that protect the highway from undesirable encroachments has, of course, been recognized for some time. A 30-year-old report states that "Parkways will benefit future generations as well as the present. Parkways have great duration, though the surface may wear out several times" (*14*).

The permanence of highways with parklike surroundings results primarily from the way in which time affects different elements of the highway and the highway environment. Pavement and buildings grow older and deteriorate according to historical time, but a landscape with a minimum of care regenerates itself by the process of nature—the cycles of day and night and the seasons. The value of having parklike landscaping along highways may become even more pronounced in the future. Some of the current practices not only tend to simplify maintenance, as noted below, but emphasize practices that may foster this regenerative aspect of natural landscaping—no-mow ground covers, small plantings that may reseed themselves, uneven rows or clumps of plantings so that replacements are not necessary (*15*, *16*).

Benefits From Scenic Enhancement

Scenic highways provide benefits to at least two groups: Highway users and nearby landowners. Esthetic landscape design typically enhances the motorists' safety and enjoyment, and stabilizes community desirability and property values. Roadside beautification is really just another way of striving for the *complete highway*—a highway that incorporates its esthetic factors in the abutting landscape corridor and thus contributes to highway safety, economy, and utility, and to the esthetic character of the corridor itself. When combined with rigorous engineering standards, highway beautification makes pleasure driving more enjoyable, but does not impair the highway's usefulness for commercial or other types of traffic.

Highway user effects

Surveys of motorists' desires show that scenic, or beautiful, highways are preferred by nearly all highway users. Some motorists have such a strong preference for scenic routes that they will travel farther or longer to traverse a scenic highway. In one survey scenery was rated as more important than travel time or distance but less important than congestion. Scenery was rated highest for pleasure driving with no particular destination and for driving to vacation destinations. Apparently, the more time a motorist has to spend on his trip, the more he is likely to select a scenic route (*17*).

Land values

The pleasure that scenic highways or parkways are meant to generate is reflected, at least in a general way, in the development of land and near these facilities in the land value. Land values provide a fairly objective measure of the economic potential of a piece of property or of an area;[4] land values can be an indicator of "all the various direct and indirect impacts".[5] They ordinarily reflect changes affecting property, whether there is a favorable influence such as a school, a park, a parkway, or a scenic highway, or whether there is an adverse influence such as a noisy, dangerous traffic arterial, or the presence of air pollution. Land values have been a useful tool in assessing the effects that water resources, urban renewal, recreation, parks, highways, and other projects have on nearby or other affected areas (*18*). Although many of these effects might have been merely attracted rather than created by the parkway or other project, there is considerable agreement that well-conceived and well-located projects may be associated with land value gains—owing to increased economies and efficiencies—that will not be offset by losses elsewhere.

Though improved esthetics or visual quality appear to be the primary purpose of scenic highways, these facilities also produce economic benefits. This generalization can be made on the basis of analogous experience with parks and open space, on the strong preference which prudent real estate investors have for pleasant surroundings, and on the persistent economic well-being which has been associated with parkways.

Parks, open space, and parkways

There is general recognition of the beneficial influence that parks and open space can have on nearby areas. The benefits of parks and open space, or least some of them, accrue to nonparticipants, to people passing by the park, and to residents living near the park who may never make any direct use of it. This park effect typically shows up in increased land values nearby. In Washington, D.C., for example, parks have been credited with enhancing nearby property values to such an extent that the resultant increase in property taxes has far exceeded maintenance and operating costs of the parks (*19*). Experience in Boston further illustrates the beneficial effect of a park or open space used in connection with a highway. Properties along the Charles River and near Jamaicaway where the parkway widens to encircle Jamaicaway have benefited; this is in contrast to areas near South Bay and Dorchester Creek where streets and highways have not been protected by sufficient space or parkland and have not benefited (*15*).

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[4] Land values tend to be more objective than some indicators, such as opinion surveys, because land values avoid most of the problems associated with interpreting the real attitudes of respondents; land values are based on verifiable contracts rather than on statements or responses that may sometimes be self-serving or otherwise inaccurate.

[5] From remarks by Allen V. Kneese, Resources for the Future, in his unpublished paper *What are We Learning from Economic Studies of Environmental Quality?*, p. 9.

Private developers make good use of the tendency of open spaces, parks, and parklike highways to enhance values of nearby land. In Los Angeles, where the Hollywood and Santa Ana freeways were constructed through areas covered by old buildings that had to be removed, plantings so improved the general tone of the neighborhoods that owners have been stimulated to renovate, reconstruct, and develop their properties (*20*). Many developers find that the income foregone by keeping certain areas untouched is more than recovered by the higher prices received for those properties that sell.[6] One developer is even going to an expense of $15,000 an acre to *wipe out commercialism.* The benefits that *a beautiful parkway* provides for his development are obvious to this developer (*21*).

The benefits realized from parks, parkways, and trees and other plantings on public space is fairly understandable. Scenery has value and can be enjoyed whether trees and plantings are owned by the public or by individuals. Trees along the highway can be enjoyed by many people and may possess even greater value than trees on private property. Such trees obviously must be located far enough away from the highway to avoid being a hazard to highway traffic.

Figure 2.—Scenic area of George Washington Memorial Parkway in Virginia.

Experience near parkways

Parkways, or scenic highways, have some of the characteristics of parks and some of the characteristics of highways. A recent study in the Washington, D.C. area has provided a comparison of land values near parkways and other highways (*22*).[7] In the period 1950–60, parkways generally outpaced nonparkways both in land value changes, adjusted for general price changes, and in the share of Washington's development of areas near parkways and nonparkways. For example, the corridors along the George Washington Memorial Parkway in Virginia (fig. 2) and the Baltimore-Washington Parkway in Maryland (fig. 3) have developed at a faster rate and have had higher increases in land value than has the corridor along Shirley Highway. Property values near parkways and other highways in the Washington, D.C., metropolitan area are shown in table 1 for the period 1950–60.

A comprehensive study of parkways and land values, completed several years ago, showed that parkways in the vicinity of New York, Boston, and Kansas City were economically sound. Land values in the areas affected, taken to be "the measure of the real estate's possible benefits to the available purchaser who can make the most profitable use of it" (*23*), increased substantially.

Experience near other scenic highways

A recent attitude survey in Chicago indicates how nearby residents feel about some of the newer highways where attention has been given to making them esthetically pleasing. In response to the somewhat leading question, "Do you consider the expressway a thing of beauty?" the responses were "yes" from 70 percent along the Edens Expressway, 100 percent along the Kennedy Expressway, and 80 percent along the Eisenhower Expressway. Open-cut areas of greenery were generally preferred to close neighbors, and the Eisenhower Expressway was considered a factor in revitalizing the slum area nearby. Some residents regarded the expressway as their park, to be enjoyed visually, even though they did not physically enter the area (*24*).

Local tax base effects

Some of the effects that the Highway Beautification Act may have on local tax bases have been referred to, at least incidentally, in the discussions of land values, business activity, and input-output model. Scenic enhancement provisions should be beneficial for local tax bases, but some adverse

Table 1.—Property values near parkways and other roads in the Washington, D.C. area—1950–60 [1] [2]

Types of highway	Adjusted value per square foot		Gain 1950–60	
	1950	1960	Dollars	Percent
Parkways:				
George Washington Memorial Parkway	$0.18	$0.81	+0.63	350
Baltimore-Washington Parkway	0.40	1.38	+0.98	245
Rock Creek and Potomac Parkway	[3] 9.14	10.35	+1.21	13
Other roads:				
Shirley Highway	0.41	0.74	+0.33	80
Seven Locks Road [4]	0.04	0.35	+0.31	775
Old Dominion Drive	0.54	1.21	+0.67	124

[1] Source: D. L. Ames and J. T. Davis, *Parkway Impact Study, An Investigation of the Effects of Parkways in the National Capital Region*, George Washington University, Washington, D.C., 1962. Property values include land and improvements. Assessments, verified by some market sales, were used in the study. Each of the study areas has a minimum extent of either six properties or 2,100 feet from the right-of-way.

[2] Types of traffic were not analyzed in this study. Research now underway, such as the "Effect of Highway Landscape Development on Nearby Property" by Franklin Institute Laboratories, a study that is part of the National Cooperative Highway Research Program, may ascertain the extent to which parkway effects are due to the absence of noisy truck traffic as well as to the presence of landscaping.

[3] Values are for 1945. Developments near the Rock Creek and Potomac Parkway appear to look to other major streets in the area for their access to community services and places of employment. The study area (from M Street to Calvert Street and from 31st Street to Florida and Connecticut Avenues) included some of the highest valued properties in the District of Columbia as well as some of the lowest.

[4] The increasing share of Washington's development which the area near Seven Locks Road experienced apparently results largely from: (1) Change from farm to vacant or residential, (2) speculative activity in anticipation of the Capital Beltway, and (3) the spatial association of residences with Burning Tree Country Club, and Bradley Boulevard.

From remarks *Toward Measuring the Intangible Values of Natural Beauty*, by Judith M. Huff and Hugh A. Johnson, U.S. Department of Agriculture, presented at annual meeting, Soil Conservation Society of America, Albuquerque, N. Mex.

[7] Although some of the growth near the Baltimore-Washington Parkway may result from such nonroad influences as [illegible] development in the direction of Baltimore, the use of the so-called participation ratio generally adjusts those nonhighway influences that were present at the beginning of the study period in 1950.

Figure 3.—Planting shrubbery along the Baltimore-Washington Parkway in Maryland.

effects may be experienced by local taxing jurisdictions where scenic easements retard development. Such restrictions may occasionally divert development to other taxing jurisdictions, but ordinarily, scenic easements will only cause whatever development that occurs to be removed farther from the highway. Some adverse effect may also result from the removal of property from tax rolls because of the acquisition of additional land for scenic strips. Both of these possible adverse effects should be minor and more than offset by general enhancement of tax rolls because of the act, but no concrete evidence of this is now available.

Junkyard control provisions are expected to result in a general improvement of local tax rolls. This is discussed later. Billboard control provisions will also have some effects on local tax rolls. Based primarily on information and comments supplied by analysts involved in State studies, it appears that local tax authorities seldom collect taxes on sign structures owned by advertisers such as motels and restaurants. The number of these signs is approximately one-half of all the signs along the highways. Property taxes ordinarily are collected for signs owned by advertising companies, but apparently not in all taxing jurisdictions. It seems obvious that no decrease in local property tax revenues will result from the loss of sign structures for which no local property taxes have been paid. In fact, if the highway payments made for sign structures are invested in some form of property that is taxable—for example, in improving the appearance of the *plant* by landscaping or by a larger on-premise sign—local property tax rolls might be benefited by the beautification program. To the extent that the removal of signs lowers revenue produced by State or local sign licenses or fees, the act may have an adverse effect. This should be very minor as these fees are usually nominal.

The effect that billboard removal might have on land values and taxes appears to be minor. Some State researchers have found that local tax assessors typically assess land on its value in a nonbillboard use so that sign rental or its loss has no effect on taxes. Although there are indications that the value of some land parcels might increase if billboards on the parcel and on land nearby are removed, there are no definitive findings yet available to support this fact.

Maintenance benefits

Good landscaping often eases maintenance problems and costs. In Ohio, "expenditures for roadside development, flattening and rounding slopes, seeding, landscaping, and erosion control have provided handsome returns in reduced maintenance expenses* * *." (*3*). The possibility that highway beautification can save money on maintenance was also referred to at a recent Ohio State University symposium. Savings in maintenance operations result from functional plantings that save mowing, reduce headlight glare, hide litter, abate noise, guide drivers, screen undesirable views, and serve as snow fences. Living snow fences have saved up to $500 per mile in maintenance costs.[8]

It is obvious that some aspects of well-landscaped highways raise, as well as lower, maintenance costs. Rest areas, for example, often create serious maintenance problems caused by vandalism.[9] Yet rest areas lower costs for such important maintenance items as litter control. This is shown by the heavy use of litter barrels at rest areas. Surveys of facilities used at rest areas demonstrate the importance of making trash barrels easily accessible—as many as 16 barrels of trash are collected each day from some rest areas. Though disposing of this volume is a major task, it is much more economical to handle rubbish in barrels than when it is scattered along the roadsides. Even if the barrels are used for domestic garbage, it is still better to have it there than in a ditch (*25*). Also, providing pleasant and neat roadsides with adequate rest areas, which are equipped with trash receptacles, should influence more motorists to keep these well-landscaped highways neat.

Benefits From Outdoor Advertising Control

Benefits from the control of outdoor advertising will accrue primarily to roadside businesses, landowners, local communities, and outdoor advertising companies. Outdoor advertising companies, for instance, will experience tangible benefits that will offset some of their losses from sign control. The industry itself recognizes that heavy concentrations of signs reduce the value of each individual unit, and that fewer signs per mile increase the profitmaking ability of each sign. The Traffic Audit Bureau, an organization supported by both advertisers and outdoor advertising companies, considers proximity to other structures a disadvantage when auditing the *space position value* of a sign. Some executives of companies using outdoor advertising have recently warned that many sign locations are becoming overcrowded and are therefore reducing the value of all signs at these locations. Also high gross receipts per outdoor advertising establishment have been associated with, but not necessarily caused by, low sign densities. The State of California, where outdoor advertising signs have been controlled for years, ranks among the States that are lowest in signs per mile of highway, but highest in gross receipts per outdoor advertising establishment.

Roadside business establishments may also experience gains from outdoor advertising control that result from a reduction in unnecessary advertising costs. Some roadside business operators use off-premise outdoor advertising signs only because their competitors do. In a study by New Mexico State University, it was found that several motel managers would be willing to remove their off-premise signs to save the amount being spent on outdoor advertising, provided that their competitors did likewise.

Landowners are also benefited by outdoor advertising control. In some places the removal of commercial signs will make the area more attractive, thus permitting higher and more profitable types of development. Also, some landowners in commercial and industrial areas, where signs are permitted, may find that their land has become more valuable as a sign site and may receive an increase in rental income. This increased value will result from the decreased supply of land for signs, and it is probable that the scarcity of land will promote signs of a higher quality to increase their effectiveness. With only a limited number of signs permitted, it would be uneconomical for an outdoor advertising company to maintain unsightly and ineffective signs. An indirect effect of the improvement in quality could be an increase in landowner income.

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[8] From remarks *Feasibility of Incorporating Natural Beauty Into Highway Design*, by Robert F. Baker, delivered before Symposium of the Ohio State University, Natural Resources Institute, Columbus, May 1966.

[9] For this vexing problem, the vandalproof comfort stations announced in *Public Works*, vol. 97, No. 3, March 1966, p. 48, may provide a solution.

The benefits that *communities* receive from outdoor-advertising control include the improved opportunities for business afforded by attractive surroundings. For example, one testifier, at the Arizona hearings on beautification standards in March 1966, stated: "Since the removal of the off-premise billboards, none of the well-established businesses in the area have felt any 'pinch.' I can honestly say that business has been *up* in our area. In all ways we business people of Sedona-Oak Creek Canyon have felt that in doing away with the billboards we have enhanced our businesses, our community, and our way of life."

Official information centers and right-of-way signs

The writers of the Highway Beautification Act of 1965 were fully cognizant of motorists' needs for information and provided for the construction of information centers, which will be beneficial to motorists, roadside businesses, local communities and even to outdoor advertising companies. Information centers, built with both private and public funds, have been important aids to the States' economies as well as new sources of business to hotels and motels.

Many advantages of information centers over off-premise commercial signs are realized by a tourist-oriented business. First, information centers may be more effective in communicating with the traveler; they enable the businessman to tell his full story about the facilities he has to offer, about his rates, and sometimes even about vacancies. Second, the cost of advertising is substantially less; example, the cost of a sign at the information center in Aspen, Colo., is $12 initially plus $24 per year, whereas costs for advertising on off-premise signs are usually much higher. Third, the advertiser does not risk losing those customers who might be opposed to signs along a highway.

In addition to information centers, official signs on the right-of-way of interstate highways, evolving from the highway beautification program, may provide tourist-oriented businesses with opportunities to communicate with motorists. A demonstration project conducted along Interstate Highway 95 in Virginia showed amply how official signs can benefit roadside businesses. The study revealed that of 578 customers, 97 percent of the gasoline customers, 89 percent of the food customers, and 93 percent of the lodging customers indicated that the specific information signs had met their needs. The researchers covered several of the signs advertising gasoline stations to test their usefulness. In almost all cases, the quantity of gasoline sold by the stations listed on the information signs was greater when the signs were exposed than when they were covered.

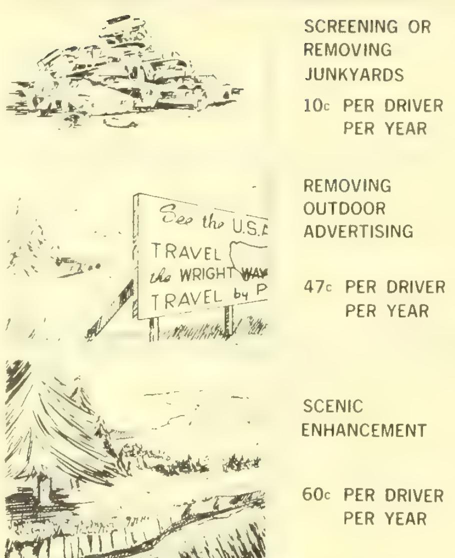

Figure 4.—Highway beautification costs over a period of 20 years.

Official information centers and right-of-way signs provide an excellent alternative source of income for outdoor advertising companies that will lose under outdoor advertising control. The manufacture, erection, and maintenance of signs and other materials at information centers, for example, can probably be accomplished best by the skills and business experience of established outdoor advertising companies. Several outdoor advertising companies have shown an interest in providing advertising services of this type or are already engaged in them. One company has recently erected a building in Iowa to provide the types of information needed by the traveler, including points of interest and information about health and emergency and personal services. Information of this type cannot readily be obtained from off-premise billboards as they now exist. The company, in suggesting the important qualifications that experienced outdoor advertising companies should have to provide information center services, stated: "The role of an experienced, private business * * * is vitally important because of the demanding service requirements of highway advertising. * * * Changes in advertising messages require personal contact with the advertiser * * * finished art, production and placement of the changed advertisement. A private advertising service company is staffed and equipped to perform these services."

The state of uncertainty that has existed in the outdoor advertising industry is another reason that investment in information centers or right-of-way signs would be advantageous. Advertising through information centers an by right-of-way signs seems to be well accepted by advertisers, by the local communit and by the traveling public. This has not bee true with off-premise signs and billboard For many years, local and State government have been enacting legislation to eliminat off-premise signs. Many garden clubs an other groups have been opposed to this typ of advertising, and advertisers have bee aware of this public reaction. By investing i information centers which are desired an needed by the traveling public, the industr could find itself in a more stable environment

Benefits From Junkyard Control

As with outdoor advertising control, th primary benefits of junkyard control ar experienced by the motoring public in th form of increased driving pleasure. However some tangible benefits will also accrue t junkyard operators, landowners, and businesses supplying goods and services for screening junkyards. A better public image resulting from junkyard screening will be the main benefit to junkyard operators. This is supported by a finding in a study, conducted by Pennsylvania State University, in which several of the larger junkyard operators interviewed generally favored the Highway Beautification Act because they felt that screening would result in better public relations.

Junkyard control provisions are also expected to enhance property values and therefore tax rolls, as mentioned previously. This is particularly true for screened junkyards. Several qualified analysts believe screening will improve neighborhoods and land values near screened yards. Relocated junkyards may result in tax gains for some jurisdictions and in losses for others. Relocation may produce a tax loss from relocated junkyards that previously paid taxes and that canno survive a move, but new activities appearing at the sites vacated by junkyards may have higher incomes and produce more taxes than the displaced junkyard operations.

Benefits Perceived Through Unit Costs

As suggested, the benefits of the highway beautification program should exceed the costs. In the absence of precise information about benefits, costs can sometimes be described or restated in terms that permit decisionmakers to compare benefits and costs in an effective way. For example, the costs of highway beautification were at one point estimated to be approximately $1.17 per driver per year, with total costs amortized over a 20-year period at 6-percent interest, compounded annually, and divided among 99 million drivers. The costs per driver per year are shown in figure 4, but should actually be somewhat less because (1) these unit costs were derived from the relatively expensive program under consideration in early 1967, and (2) in the future there will be more than the present 99 million drivers to share these

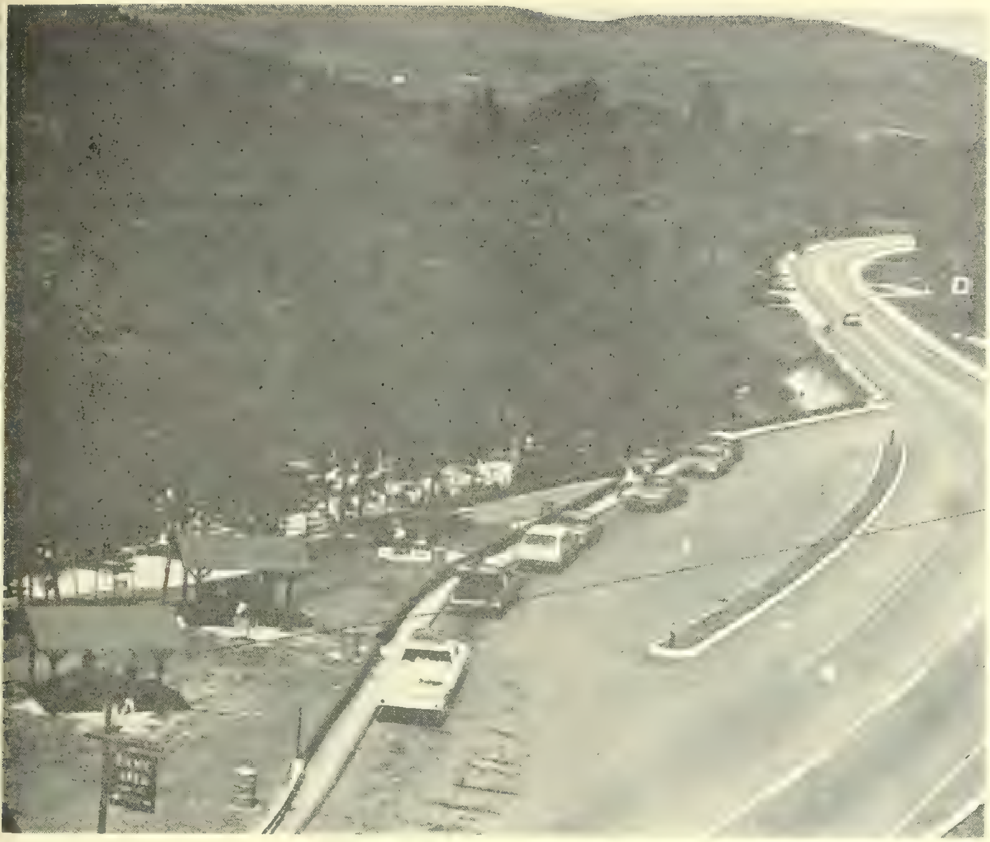

Figure 5.—Snow Falls rest area on Route 26, West Paris, Maine.

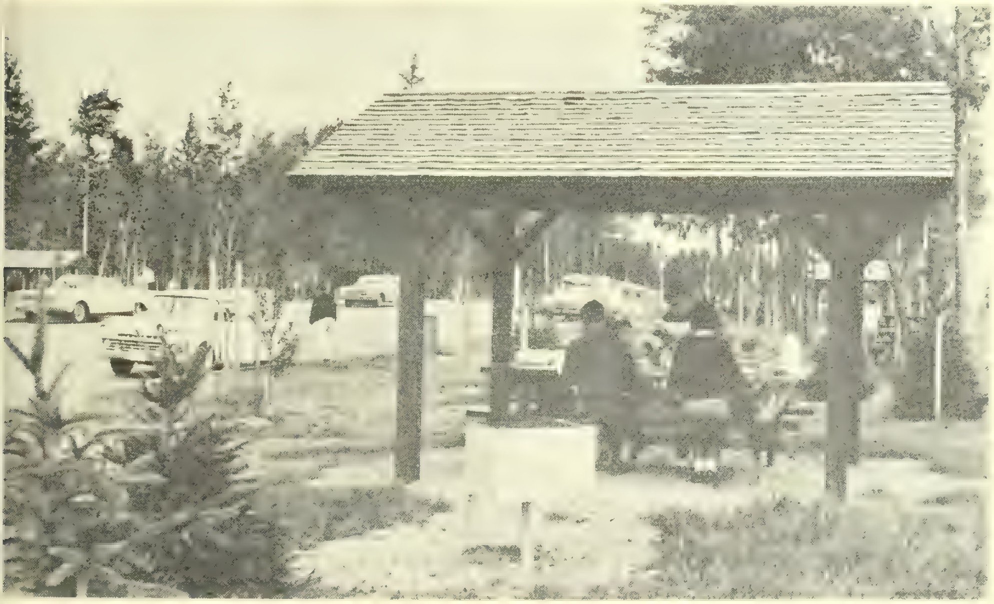

Figure 6.—Rest area on I–95, southbound, near Pittsfield, Maine.

costs. Also, unit costs should be substantially less if passengers as well as drivers were considered.

Rest area benefits

The benefits derived from rest areas are substantial, but whether they are as great as the cost is a matter that must be considered. Under the highway beautification program, rest areas on the Interstate system will cost about $150,000 each to construct. Based on incomplete information, the 2,500 rest areas along the Interstate system may attract between 5 and 14 percent of the traffic passing by these facilities. If the percentage of vehicles stopping is as low as 5 percent, costs would amount to less than 5 cents per car for each visit, assuming a 20-year life for the rest areas. Examples of the use of rest areas by the traveling public are shown in figures 5 and 6.

Some insight may be gained concerning the value visitors place on these stops by considering the amount of time they spend at rest areas. Very incomplete data indicate that visitors usually spend about 15 minutes each time they stop at rest areas. Motoring for pleasure has sometimes been valued at about 36 cents per person per hour, a figure derived generally from the vehicle operating cost divided among the people in the vehicle traveling for pleasure (*11*).[10] Assuming that people traveling for pleasure do value motoring at 36 cents an hour, it seems that they are placing considerable value on rest area stops. Each time a pleasure traveler stops for 15 minutes, he is apparently foregoing, or at least postponing, travel that he values at about 9 cents.

The Cost of Pleasure Driving

Considering the cost of pleasure driving provides some insight into the benefits yielded by highway beautification. Motor-vehicle-use studies have shown that at least 12 percent of the automobile travel in the United States is for pleasure. This means that approximately 90 billion vehicle-miles were driven for pleasure in 1966 (12 percent of the 750 billion vehicle-miles traveled on all highway systems) when traffic volumes were considerably lower than they are at present. If each motorist was aware that his variable or operating costs were approximately 6 cents for each mile traveled, it follows that the motorists traveling the 90 billion vehicle-miles for pleasure placed a high value on this travel by paying more than $5 billion per year to engage in pleasure driving. A comparison between this cost and the estimated yearly costs for beautifying the highways would not indicate whether the amount planned for highway beautification is appropriate; but the magnitude of the cost of this pleasure-driving does indicate the resources motorists are willing to commit to pleasure driving.

Summary

The expense of highway beautification has received much attention, and the study completed cooperatively by the States and the Bureau of Public Roads has provided specific information about the costs of the program. Other material pertaining to costs, including estimates of indirect effects, has been made available to the Congress by various groups, some of which are affected adversely by the act.

The discussion in this article is limited to the benefits of highway beautification. It is intended to provide information to complement that already available on costs. Hopefully, this description of some of the benefits of highway beautification will be useful to decisionmakers in weighing the costs and benefits of the program.

Benefits of the program are substantial and significant, but they are also largely intangible and unmeasurable. Their illusive nature makes it difficult to meaningfully compare them with the costs of the program. The following generalizations can be made about these benefits:

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[10] The 36 cents-per-hour estimate may understate the value motorists place on pleasure driving, as (1) it was based on a traffic speed of about 25 m.p.h., which is lower than current traffic speeds, and (2) it was assumed that four people were in each car rather than the three or fewer that State-Bureau studies now show to be typical for pleasure travel.

- Highway users will experience substantial benefits from highway beautification in increased comfort, convenience, pleasure, and safety.
- The highway beautification program is a part of and is fostering the general interest in a higher quality of living now so apparent in this country.
- Highway beautification will generate economic activity because of the spending for such beautification measures as nursery products and informational signs, and the enhanced opportunities for economic growth in areas made attractive for tourists or other consumers.
- The economic life of a highway can be prolonged by scenic enhancement. The enduring values associated with parkways now several decades old, demonstrate the wisdom of surrounding highways with landscaping that will endure rather than with manmade structures that may soon become obsolete.
- Outdoor advertising controls should result in fewer but more attractive and profitable signs.
- Outdoor advertising controls may reduce advertising costs—as, for example, by roadside businesses now using outdoor advertising only because their competitors do.
- Junkyard screening should benefit junkyard operators by providing an improved public image.
- Some landowners will benefit through higher land values.
- Official information centers and right-of-way signs have benefited motorists, roadside businesses, and local communities.
- The need to provide advertising services at information centers offers significant economic opportunities for outdoor advertising companies.
- In addition to the benefits of highway beautification, there are costs from not beautifying highways. These include roadside blight and all the problems of blight—highways that become obsolete prematurely, highways that are unsafe and confusing for motorists, and increasing handicaps in competing for tourist trade in the world market.

When these benefits expected from highway beautification are considered along with the costs and disadvantages, decisionmakers may be aided in determining the future of the program.

Studies of the Economic Impact of the Highway Beautification Act of 1965—Listed by State

Alabama

Capital Income Effects of Signboard Regulation, Control, and Removal on Selected Kinds of Businesses and Land Segments Located on Interstate and Primary Highway Systems in Alabama, University of Alabama.

California

Impact and Implications of Highway Beautification Act of 1965 on Motels in the City of Chico, California, California Division of Highways.

Connecticut

Environmental Appearance as a Factor in Highway Design, Yale University.

Georgia

A Study to Examine Some Critical Areas in Aesthetics and Economics as These Topics Relate to Land and Land-Based Structures, Georgia State College.

Indiana

A Study of Some Effects of Junkyard Screening and Billboard Density, Purdue University.

Kentucky

Economic Impacts of Highway Beautification Act in Kentucky, Spindletop Research Center.

Maine and New Hampshire

Highway Beautification Study—Maine and New Hampshire, Wilbur Smith and Associates.

Missouri

Estimates of the Impact of Sign and Billboard Removal Under the Highway Beautification Act of 1965, University of Missouri.

Nebraska

Economic Impact of Highway Beautification (*Economic and Legal Problems Related to Roadside Beautification and Recreational Development Along Nebraska Interstate 80*), University of Nebraska.

New Jersey

Identification and Estimation of Economic Benefits from Scenic Enhancement of New Jersey Highways, Rutgers University.

New Mexico

The Effect of Highway Billboard Advertising Upon Selected Southern New Mexico Motels and Their Guests, New Mexico State University.

New York

A Study To Investigate the Impact on the Outdoor Advertising Industry in Reaction to the Highway Beautification Act of 1965, Rennselaer Polytechnic Institute.

Ohio

Comprehensive Evaluation of Highway Sign and Billboard Regulations, University of Akron.

Pennsylvania

Selected Effects of the Highway Beautification Act of 1965, Pennsylvania State University.

Tennessee

The Economic Impact of the Highway Beautification Act on the Outdoor Advertising Industry, Landowners, and Selected Scenic Attractions of Tennessee, University of Tennessee.

Methods Used by Tennessee Motorists to Select and Locate Lodging and Automotive Services, Memphis State University.

Texas

Economic Effects of the Highway Beautification Program, Texas A&M University.

Washington

Impact of the Highway Beautification Ac on Outdoor Advertising Firms in Washington State, University of Washington.

Peripheral Land Study: Economics, Esthetics, and Highways, University of Washington.

Factors Influencing Patrons to Stop a Selected Types of Motels, University of Washington.

West Virginia

The Impact of Highway Beautification on the Outdoor Advertising Industry in West Virginia, West Virginia University.

Wyoming

Attitudes of Highway Oriented Businesses in Southern Wyoming Toward Highway Billboard Advertising, University of Wyoming.

Traveler Attitudes Toward Highway Billboard Advertising: A Survey of Selected Wyoming Motel Patrons, University of Wyoming.

Nationwide

A Survey of the Standard Poster Outdoor Advertising Industry, Memphis State University, Tenn., and Texas A&M University, Tex.

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(*1*) *The Economic Impact of the Highway Beautification Act of 1965,* by the staff of the Office of Research and Development, Bureau of Public Roads, February 1967.

(*2*) *The Natural Environment,* by Joseph Fisher, The Annuals of the American Academy of Political and Social Science, in *Social Goals and Indicators for American Society,* vol. 1, May 1967, pp. 127–140.

(*3*) *Highway Scenic Beauty Doesn't Just Happen,* by Wilbur J. Garmhausen, Public Works, vol. 97, No. 9, September 1966, p. 124.

(*4*) *Impact of Improved Highways on the Economy of the United States,* Stanford Research Institute, Menlo Park, Calif., 1958, p. 40.

(*5*) *How a Strike Looks to a Computer,* U.S. News and World Report, vol. LIX, No. 20, Nov. 15, 1965, pp. 98–100.

(*6*) *The Conference Call,* Beauty for America, proceedings of the White House Conference on Natural Beauty, Washington, D.C., May 24–25, 1965, p. 569.

(*7*) *A New Sense of Urgency,* by Laurance S. Rockefeller, General Electric Forum, vol. VIII, No. 4, October–December 1965, p. 5.

(*8*) *Make a Charming Entrance for Your City and Draw Tourist Dollars,* by Lee F. Betts, The American City, January 1966 pp. 94–95.

(Continued on p. 17)

Highway Interchange Locations for Churches—A Pilot Study in Washington, D.C.

Y THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by MARTIN M. STEIN, JOHN YASNOWSKY, JR., and
FLOYD I. THIEL,[1] Economists, and MICHAEL R. HORWITZ,
Sociologist, Economics and Requirements Division

The optimal use of the highly valued land in the vicinity of highway interchanges has been a subject for research and discussion for many years. The pilot study described in this article sought to provide some insight into whether churches and similar social institutions are among those land uses that can profit from an interchange location and in turn contribute to the efficient and orderly development of the interchange area. The scope of the pilot study was limited to learning the attitudes toward interchanges of church spokesmen in the Washington metropolitan area and to determining the extent and nature of previous relevant research. Information was obtained by conducting a literature search, by interviewing spokesmen for 35 of the 36 churches located at interchanges along the Capital Beltway, I–495, and by interviewing several church location planners who are responsible for selecting suitable sites for future church buildings.

From the findings of the study it is suggested that churches often can benefit from interchange locations. This results partly because modern churches are largely automobile oriented, and accessibility to the motorist is important. Interchange areas might be more appropriate for churches in which the members reside some distance away than for those churches in which the members reside in the immediate neighborhood. Church location planners generally favored sites near interchanges because of two important interchange characteristics—visibility and accessibility. Those planners who did not favor interchange locations felt that land costs were too high or that it was more important to be located in the center of a residential neighborhood.

Regardless of the opinions expressed by pastors and church-location planners, one fact was evident: more interchanges on the Capital Beltway, I–495, contain a church, than a filling station, a restaurant, a motel, a shopping center, or a school.

Introduction

THE ACTIVITIES that are to be located near highway interchanges must be deternined carefully. Some activities can benefit rom being near a highway interchange, and, t the same time, be beneficial to the interhange area; other activities cannot gain ufficiently to justify paying the high land osts often associated with interchange locaions. Certain types of activities may even e impaired if they are located near an interhange or may be harmful to the interchange.

Progress has been made in ascertaining land ses that are suitable for interchange areas nd in determining those that are compatible vith one another near interchanges. Establishments providing highway services, for xample, have an important role at many nterchange locations. Other commercial and ndustrial activities also appear appropriate n many interchange areas, as do residential ctivities and others, provided adequate recautions are taken to assure compatiblity etween them and the interchange.

In studies of interchange development, and in practice, some attention has been given to the suitability of interchange areas for churches (and schools). Around the Capital Beltway in Washington, D.C., for example, 36 churches have been counted within a half mile of 21 of the 36 interchanges. To learn more about the suitability of interchange locations for churches, a pilot study, described in this article, was conducted in August 1967.

Scope of Study and Methods Used

In the pilot study, data from several sources were used. From a windshield survey of general land uses along the Capital Beltway, I–495, the number and location of churches were determined. The churches included in the survey are listed in table 1 and their locations are shown in figure 1. Telephone and personal contact with spokesmen for these churches and with 10 church location planners provided primary information for the study. In addition, information was obtained from a number of secondary sources, which are listed in the bibliography *(1)*–*(35)*.[2] Although information was gathered from all the churches located within a half mile of the beltway interchanges, it is quite generalized—much less comprehensive than would be needed for a case study, for example. The time and resources available usually permitted only one unstructured interview with a spokesman from each church in the study group.

The extent to which respondents may have reflected all views of their congregations, or even a consensus, is not known. Although interviews were sought with the pastors of the churches, other well-informed representatives sometimes provided information in the pastor's absence. Even so, it is possible that these spokesmen were not aware of all the opinions that the church members might have concerning the suitability of interchange areas for church locations. Highway noise during church services, for example, might seem less of a problem to a pastor with an active role in the service than to a member of the congregation who is primarily a listener.

Trends in Church Membership

An increasing number of American people have become affiliated with religious institutions. Church membership increased from 47 percent of the U.S. population in 1930, to 57 percent in 1950, and to 64 percent in 1965.

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[1] Floyd I. Thiel is the leader of the Economics Task Group, vhich focuses on the study of the economic effects of highway nvestment. The objective of this research group is to define he essential factors that stimulate economic activity and hange land-use functions adjacent to new highway facilities.

[2] Italic numbers in parentheses indicate references listed in bibliography on page 16.

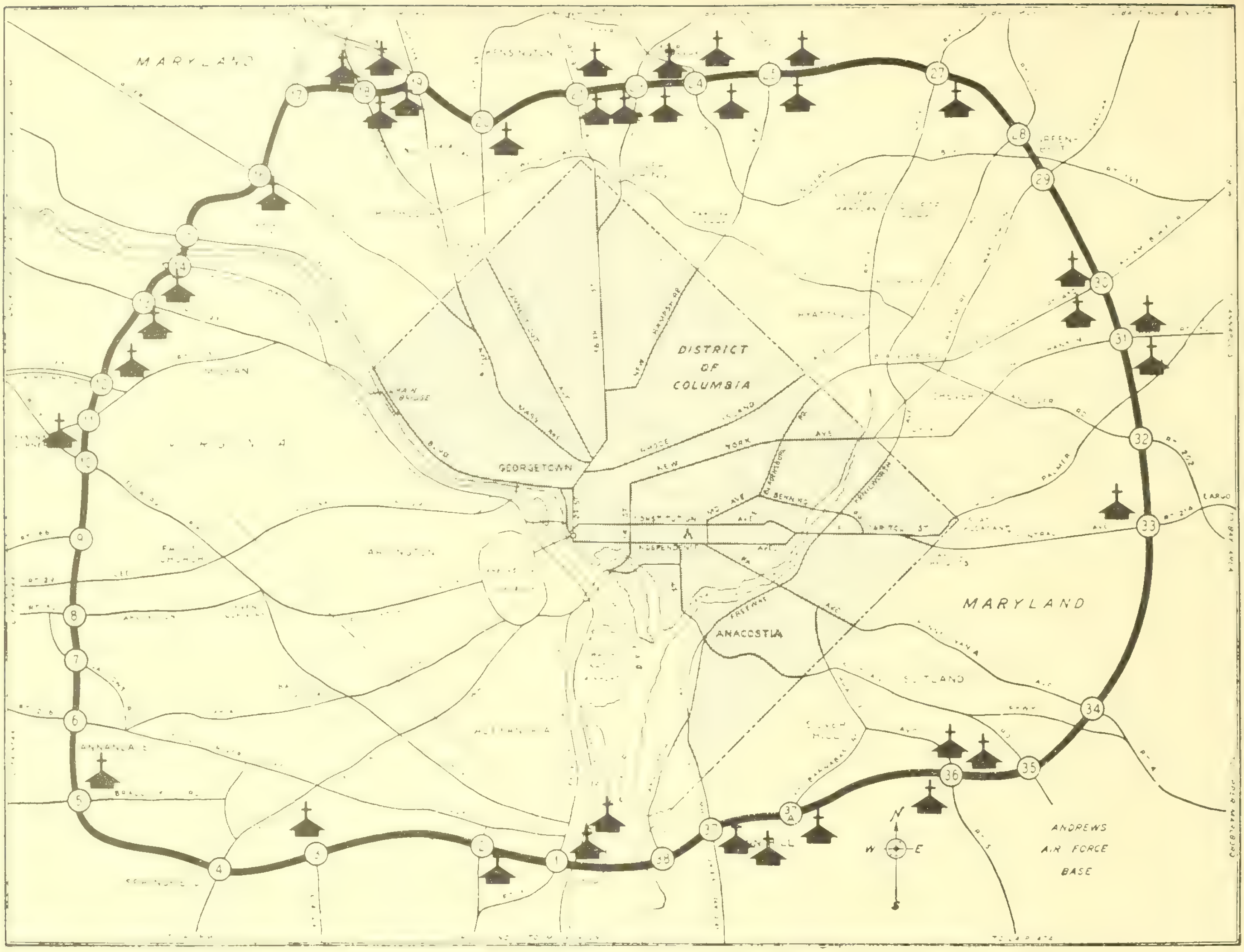


Figure 1.—Locations of 36 churches near interchanges of the Capital Beltway, I–495, surrounding Washington, D.C.

as shown in figure 2. The number of churches has also been increasing at a faster rate than U.S. population, though not as fast as church membership. The trends in the growth of churches and of church membership from 1940 to 1963 are shown in figure 3. The slower growth in the number of churches apparently reflects the tendency for individual church units to serve more members as time goes by. Yet the construction of religious buildings is big business, over $1 billion a year (*1*), and careful planning of this investment is justified.

In the Washington metropolitan area, a similar growth in church membership and in the number of churches is reflected. During the 1950–60 period, Protestant church membership increased nearly 60 percent and the number of Protestant churches increased 40 percent, while population in the Washington metropolitan area increased only 35 percent. From 1960 to 1965, however, the population grew at a more rapid rate than the number of Protestant churches, as shown in figure 4. This trend like the trend illustrated in figure 3 also shows a tendency for churches to serve more members. The information included here on membership and number of churches is confined to the churches associated with the Council of Churches of greater Washington, D.C., which represent approximately 25 percent of the total church membership and 33 percent of the total number of churches in the Washington metropolitan area.

When the Washington metropolitan area is considered by political subdivisions, certain variations are found among the churches studied. From the period 1960–65, the District of Columbia experienced not only a loss in the number of churches but even more of a loss in the number of church members. The Maryland and Virginia suburbs, on the other hand, experienced an increase both in church membership and in the number of churches, as shown in figure 5. These variations may be due to different population growth rates—District of Columbia population is growing at a slower rate—and to the fact that no data are available for some denominations such as the African Methodist Episcopal, and African Methodist Episcopal Zion.

Criteria for Church Site Selection

Insight into the suitability of interchange areas for church activities can be gained by considering some of the criteria for church location and determining whether they are satisfied by interchange areas. Several of the criteria for church sites that seem to be generally satisfied in interchange areas are visual prominence, vehicle accessibility, and easily identified address. Other criteria which interchange areas generally fail to satisfy include pedestrian accessibility—perhaps becoming less important—site location near the center of the membership or neighborhood, and site location removed from the area of high land values. Interchange areas seem no better or worse than other areas in meeting some of the requirements for church sites, which include: (1) Adequate parking facilities, (2) proximity to a shopping center (3) suitable zoning, and (4) a suitable lot, considering esthetics, utilities, size, shape, and slope.

The relative importance of the various location characteristics or criteria depends largely

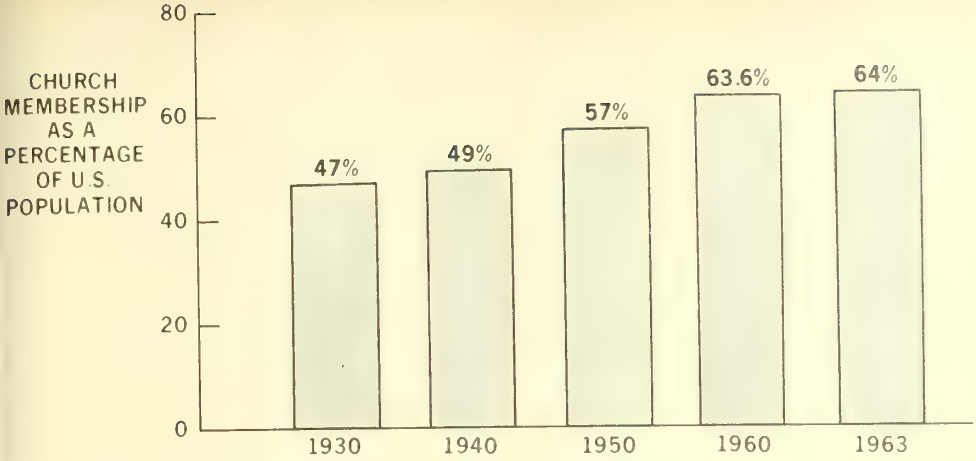


Figure 2.—Church membership as a percentage of U.S. population, 1930–63.

on the type of church. A church that draws membership from a wide area will emphasize vehicle accessibility, parking facilities, and visual prominence, whereas a neighborhood church that draws membership from a local area has, in the past, favored locations with easy access for pedestrians.

The different factors to be considered in selecting a church location often complement each other. For example, a church near a shopping center or a school may be able to benefit from good accessibility and the abundant parking typical at shopping centers and schools. Even the terrain can complement the location. Level slopes, lower construction costs, and high land provides visual prominence and, possibly, improved drainage.

On the other hand, the characteristics of a location may be conflicting. For example, good accessibility means being close to heavily traveled routes where noise and other traffic disturbances may be inherent. The same is true of the proximity of shopping centers, particularly for evening activities during the week. Visual prominence may result in higher construction cost, and close proximity to a residential area may conflict with local zoning requirements.

Experience of Church Location Planners in Washington, D.C.

Church location planners, primarily clergymen with training in planning, architecture, and administration, were interviewed during the pilot study. These specialists, who are responsible for recommending or selecting suitable sites for future church buildings, have the experience necessary to evaluate interchange locations as possible church sites. Their opinions varied as to the desirability of locating a church building near a highway interchange. The majority agreed that the visibility and accessibility afforded by an interchange site were important considerations. One planner illustrated his view of the importance of church visibility by stating, "Where a sign is located to direct motorists to the church, this is where the church should be." However, a few took the position that visual prominence could be obtained equally as well at sites away from the interchange. Some also contended that an interchange location was not nearly as accessible as a site in the center of a residential neighborhood. They pointed out that interchanges may divide the residential development into four quadrants so that a church located near the interchange facility may be on the outer edge of its actual and potential membership rather than in the center.

The generally high cost of land was one of the reasons given as to why interchange sites are not always considered the most desirable for churches. Land costs were considered by nearly all of the planners to be an important factor in site selection; consequently, planners seek to recommend locations before new development causes land prices to rise—long before actual church construction begins.

At the outset of this pilot study it was hypothesized that interchange locations might not be suitable as church sites because of the noise, fumes, and other undesirable characteristics often associated with heavy traffic. However, the discussions with the church planners indicated that traffic noise and fumes ordinarily are not troublesome, because most of the new church buildings are air-conditioned and outside interferences become less of a problem. Church activities that are conducted out-of-doors are generally held at the rear of the building where there is some protection from highway noise and fumes.

Although most of the church location planners preferred quiet residential areas for church sites, some favored sites in commercial areas along major highways or near interchanges. In addition to being visible to a larger number of passing motorists, a commercial area permits the joint use of parking

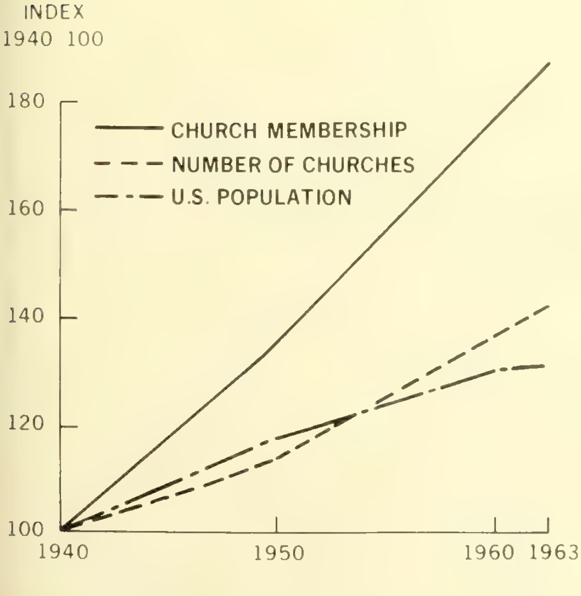


Figure 3.—National trends in church and church-membership growth 1940–63.

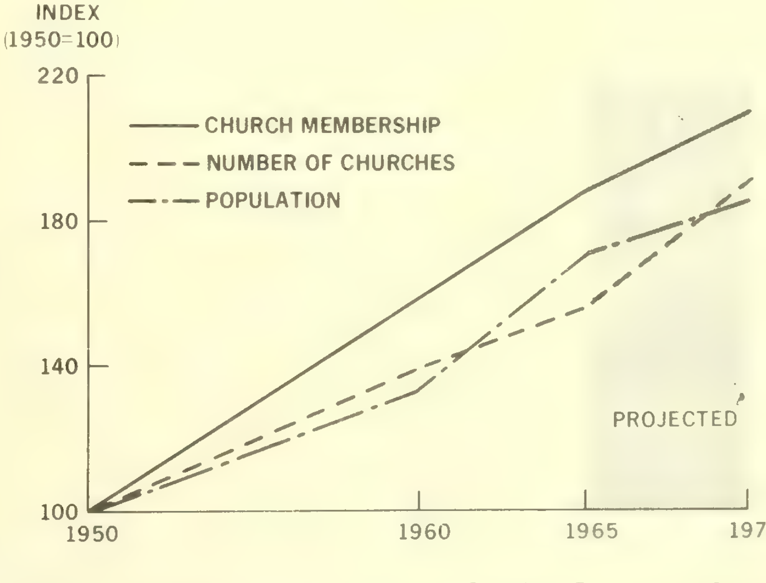


Figure 4.—Washington area Protestant churches, Protestant church members and population, 1950–70.

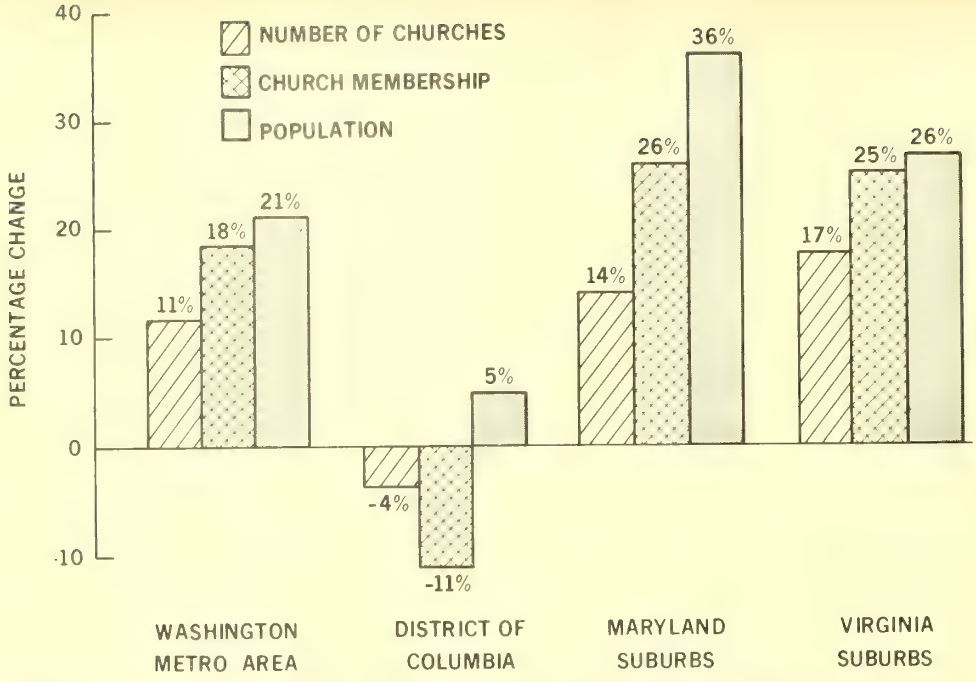


Figure 5.—Changes in Protestant churches and church membership, Washington metropolitan area, 1960–65.

Table 1.—Churches located within half mile of interchanges of Capital Beltway, I–495 [1]

Name of church	Interchange number	Feeder road
Abundant Life Independent	1	U.S. 1.
St. Mary's Roman Catholic	1	U.S. 1.
Church of God of Prophecy—Independent	2	Va. 241.
United Pentecostal	3	Va. 613.
Ravensworth Baptist	5	Va. 650.
Providence Baptist	10	Va. 7.
Latter Day Saints	12	Dulles Airport Road
St. Luke's Roman Catholic	13	Va. 193.
McLean Bible	13	Va. 193.
Concord Methodist	16	Md. 190.
St. Jane Frances de Chantal Roman Catholic	18	Md. 187.
North Bethesda Methodist	18	Md. 187.
St. Luke's Episcopal	18	Md. 187
Temple Hill Baptist	19	U.S. 240
North Chevy Chase Christian	20	Md. 193.
Calvary Lutheran	21	Md. 97.
Montgomery Hills Baptist	21	Md. 97.
Christ Congregational	23	U.S. 29
Marvin Memorial Methodist	23	U.S. 29.
Knox Presbyterian	23	U.S. 29.
Temple Israel	24	Md. 193
St. Bernadette's Roman Catholic	24	Md. 193.
Good Sheperd Methodist	25	Md. 650.
Our Savior Episcopal	25	Md. 650.
North College Park Pilgrim	27	U.S. 1.
First Baptist of Carrolton	30	Md. 450.
St. Christopher's Episcopal	30	Md. 450.
Grace Presbyterian	30	Md. 450.
Ebenezer Methodist	31	U.S. 50.
Ridgely Methodist	33	Md. 214.
Church of the Nativity Episcopal	36	Md. 5.
Church of St. Philip the Apostle Roman Catholic	36	Md. 5.
Latter Day Saints	36	Md. 5.
Henson Valley Christian	37	Md. 210.
Forest Heights Baptist	37	Md. 210
St. Ignatius' Roman Catholic	37A	Md. 414.

[1] Survey made in 1967

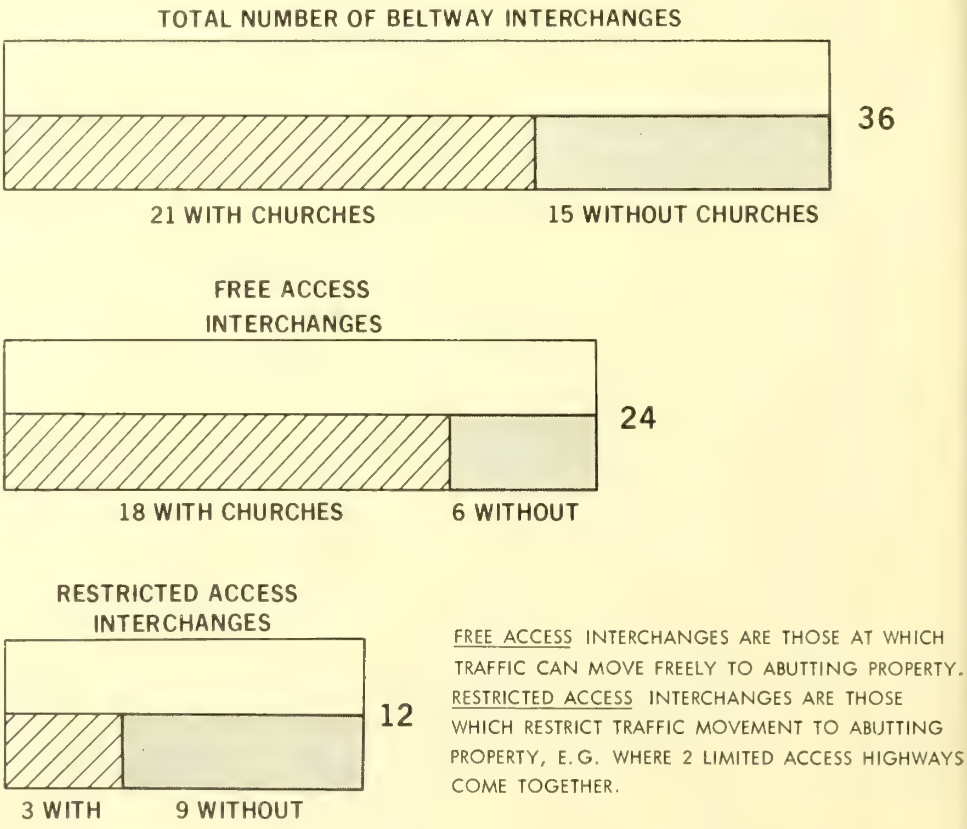
facilities owned by the commercial establishments. Nearly all the church location specialists agreed that consideration should be given to a site that permits Sunday parking on facilities used for other purposes during the week.

Several planners spoke of a planning con cept in which churches and similar instit tions are used as buffers between the con mercial developments at interchange faciliti and the residential developments farthe back. Such an arrangement, of course, woul provide some of the benefits of both resi dential and commercial locations. Also, i several churches were grouped in this fashion they might be able to realize some economie through the use of their combined facilities.

The consensus of the planners was tha churches are becoming larger, that fewe members are walking to church, and tha members are driving longer distances, partl because of improved highway transportation However, the planners felt that highways wer not an influence in increasing membershi even though the highways sometimes allow a church to retain its members after they hav moved to other neighborhoods.

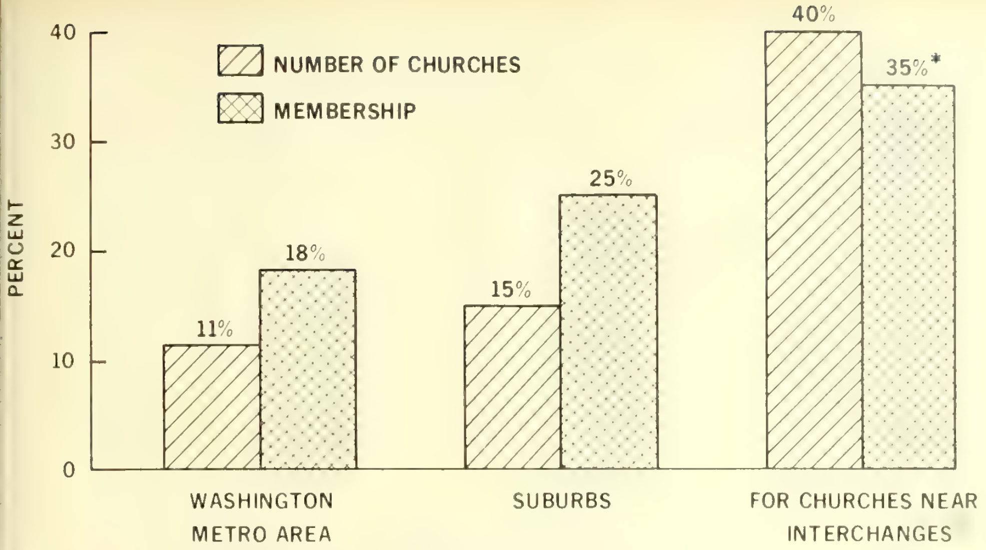
Churches on the Capital Beltwa

There are 36 churches within a half mile o the interchanges of the Capital Beltway, I–495. Information was gathered from onl 35 of the churches, as several attempts t contact one store-front church in Virginia were unsuccessful. Most of the 36 churches were located there before the beltway was constructed. In fact, only six churches have been located at beltway interchanges since it opened in 1964. Seven other churches were constructed in the early 1960's when only general information was available about the



FREE ACCESS INTERCHANGES ARE THOSE AT WHICH TRAFFIC CAN MOVE FREELY TO ABUTTING PROPERTY. RESTRICTED ACCESS INTERCHANGES ARE THOSE WHICH RESTRICT TRAFFIC MOVEMENT TO ABUTTING PROPERTY, E.G. WHERE 2 LIMITED ACCESS HIGHWAYS COME TOGETHER.

Figure 6.—Church locations by type of interchange, Capital Beltway, I–495.



* REPRESENTS 67% OF SAMPLE SIZE IN 1960

Figure 7.—Changes in Protestant churches and church membership, Washington metropolitan area, suburbs, and interchange areas, 1960–65.

beltway location. The dates that original church buildings were constructed within a half mile of highway interchanges are shown in table 2. Related highway events, such as the beltway opening, are also listed in the table. The number of new churches at highway interchanges varies between zero and three each year. Information about the general location of the beltway was available for some time before the actual opening in 1964. In Virginia, for example, six of the eight churches surveyed were constructed after acquisition for the beltway right-of-way began in January 1958, and five were constructed after December 1961, when portions of the Capital Beltway were opened to traffic.

The 36 churches are located at 21 of the 36 beltway interchanges, as stated previously. Of the 24 *free access* interchanges—where traffic can move freely to abutting property by leaving the beltway at the interchange—18 have one or more churches. Of the 12 *restricted access* interchanges—where traffic can move from one highway to another but where movement from the beltway to abutting property is restricted—only three have churches located within a half mile. Thus, as shown in figure 6, three-fourths of the *free access* interchanges have churches nearby.

Experience of Churches Near Beltway Interchanges

The information obtained about the churches near the interchanges of the Capital Beltway was limited and incomplete; yet some of it might provide an insight into the suitability of interchange locations for churches. Membership trends, for example, as well as church improvements, may be affected by a good or a poor highway location.

Churches near the beltway interchange are experiencing noticeable increases both in the number of churches and in the number of church members. Because of the absence of an appropriate group of churches that could be used for comparison, the influence of beltway interchanges on this growth cannot be established conclusively; but compared with changes in the Washington metropolitan area and in the Maryland and Virginia suburbs, churches near Beltway interchanges, during the past 5 years, have experienced the changes shown in table 3. This information was derived from records compiled by the Council of Churches of Greater Washington. The information covers most of the Protestant churches in the Washington area; much of the same information is depicted graphically in figure 7.

Spokesmen for the 35 churches surveyed generally believe that the Capital Beltway has no adverse effect on the activities of their churches. Whereas 10 of the 35 respondents admitted there was some traffic noise, 25 reported that there was no noise or at least that noise was no problem. These overall opinions of the beltway's effect on church activities are listed in table 4. As shown for the 35 churches, 12 spokesmen reported that the beltway had no effect, 21 reported a favorable effect, and 2 reported the effect to be unfavorable.

It is understandable why church spokesmen gave varied reports on the suitability of interchange locations for churches. Churches have varying needs for such highway interchange characteristics as visibility and accessibility. Some churches serve an entire community or region and are sometimes regarded—and regard themselves—as communitywide or regional churches. Other churches, regarded more as neighborhood churches, have their members predominantly in the neighborhood or in a portion of a large community. Whether a church should be regarded primarily as a neighborhood church or a church serving a large area such as a community, depends generally on how widely the membership is diffused, that is, how far members typically travel to church.

For the simple analysis described here, churches were classified generally as community churches—(1) if approximately 25 percent or more of the members travel to church by the beltway, and/or (2) if the associated denomination or brotherhood accounts for about 2 percent or less of the congregations in the Washington area. The classification is based on a preliminary analysis in which it was determined that about 2 percent may be a breaking point between those groups with approximately 25 percent or more of the members using the beltway for church travel. Applying these

Table 2.—Churches constructed in interchange areas of Capital Beltway, I–495, during different phases of highway construction

Phase of highway construction	Year	Churches within half mile of interchanges
		Number
Right-of-way acquisition started	Before 1958	17
	1958	2
Beltway partially open	1959	2
	1960	0
	1961	2
Beltway opening	1962	3
	1963	2
	1964	1
	1965	3
	1966	2
	1967	1
Total		[1] 35

[1] Information from one of the 36 churches could not be obtained.

Table 3.—Changes in number of churches and church members in different sections of Washington metropolitan area, 1960–65

Location	Change in number of churches	Change in number of members
	Percent	*Percent*
Near beltway interchanges	+40	+35
Maryland and Virginia suburbs	+15	+25
Washington metropolitan area	+11	+18
District of Columbia	−4	−11

Table 4.—Effect of Capital Beltway on activities of 35 churches, according to church spokesmen

Beltway effect	Opinions of church spokesmen
	Number
None	12
Very favorable	10
Favorable	6
Slightly favorable	5
Slightly unfavorable	2
Unfavorable	0
Very unfavorable	0

criteria to the Washington metropolitan area resulted in classifying most Catholic, Methodist, and Baptist groups as neighborhood churches and Latter Day Saints, Pentecostal, and Pilgrim congregations as communitywide or regional churches. There are obviously exceptions. For example, two Methodist churches and one Lutheran church in interchange areas seem to be community churches and some of the Latter Day Saints churches are acquiring characteristics of neighborhood churches. The history of a particular congregation and how the church regards itself are considered a part of the criteria. Figure 8 and tables 5 and 6 provide additional information on the denominations of beltway interchange churches and on the classification of community and neighborhood churches.

Of the 35 churches surveyed, 12 can be regarded as community or regional churches and 24 as neighborhood churches. Spokesmen for the 12 community churches apparently had a higher opinion of the beltway's usefulness than spokesmen for the 23 neighborhood churches. The differences in the opinions expressed by church spokesmen concerning beltway effects on church activities are listed in table 7.

Highway interchange areas can apparently meet the site needs of some but not all churches. This fact is based on the experience cited by representatives of the 35 churches surveyed, on the comments of selected church planners, and on selected literature pertaining to church planning. Table 8 lists some of the site requirements of neighborhood and community churches and refers to the apparent ability of interchange areas to meet these needs. As can be seen, interchange areas appear to be more suitable for communitywide or regional churches than for neighborhood churches.

Experiences of Individual Churches

Like other activities near interchange areas, churches often have been able to use inter-

Table 5.—Denominations of community and neighborhood churches at interchanges of Capital Beltway, I-495

Denomination	Number of churches at interchanges	Number classified as community churches[1]	Number classified as neighborhood churches
Roman Catholic	6	0	6
Methodist	6	2	4
Baptist	6	0	6
Episcopal	4	1	3
Independent	3	2	1
Mormon	2	2	0
Presbyterian	2	1	1
Disciples of Christ	2	0	2
Hebrew	1	1	0
Lutheran	1	1	0
Pilgrim	1	1	0
Pentecostal	1	1	0
United Church of Christ	1	0	1
Total	36	12	24

[1] Churches were classified as community churches if (1) approximately 25 percent of the members travel to church via the beltway, and/or (2) the denomination or brotherhood accounts for about 2 percent or less of the congregations in the Washington area. The history of a particular congregation and whether it considers itself a community or neighborhood church were also considered in the classification.

Table 6.—Relation of denominations to beltway-interchange churches and to Washington area churches

Denomination	Percent of churches near Capital Beltway[1]	Percent of all churches in Washington area
	Percent	*Percent*
Catholic	17	9
Methodist	17	15
Baptist	17	11
Episcopal	11	9
Independent	8	20
Mormon	6	1
Disciples of Christ	6	2
Presbyterian	6	7
United Church of Christ	3	2
Pilgrim	3	0.4
Pentecostal	3	0.1
Lutheran	3	6
Hebrew	3	1

[1] Percentages have been rounded and do not total 100 percent.

Table 7.—Effect of Capital Beltway on church activities, according to church spokesmen

Beltway effect	Opinions of community church spokesmen[1]	Opinions of neighborhood church spokesmen
	Number	*Number*
Favorable	12	9
Unfavorable	0	2
None	0	12
Total	12	23

[1] Churches were classified as community churches if (1) approximately 25 percent of the members travel to church via the beltway, and/or (2) the denomination or brotherhood accounts for about 2 percent or less of the congregations in the Washington area. The history of a particular congregation and whether it considers itself a community or neighborhood church were also considered in the classification.

Table 8.—Apparent capability of interchange areas to meet needs of church sites

Church site needs	Extent to which suburban interchange areas serve church site needs
Neighborhood church:	
Visual prominence	Good.
Vehicle accessibility (local)	Fair.
Adequate parking	Undetermined.
Out of high value area	Poor.
Near community center	Poor.
Pedestrian accessibility	Poor.
Community church:[1]	
Visual prominence	Good.
Vehicle accessibility (all traffic)	Good.
Adequate parking	Undetermined.
Easily identified location	Good.

[1] Churches were classified as community churches if (1) approximately 25 percent of the members travel to church via the beltway, and/or (2) the denomination or brotherhood accounts for about 2 percent or less of the congregations in the Washington area. The history of a particular congregation and whether it considers itself a community or neighborhood church were also considered in the classification.

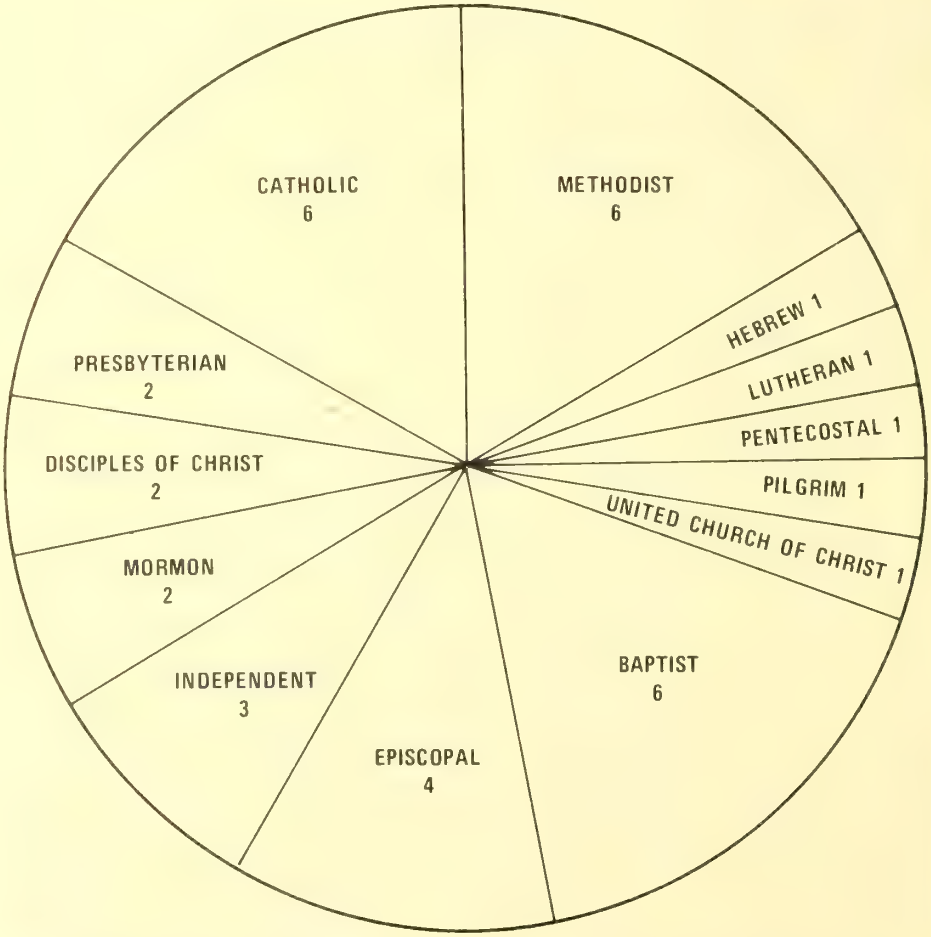


Figure 8.—Denominations of churches at interchanges of Capital Beltway, I-495.

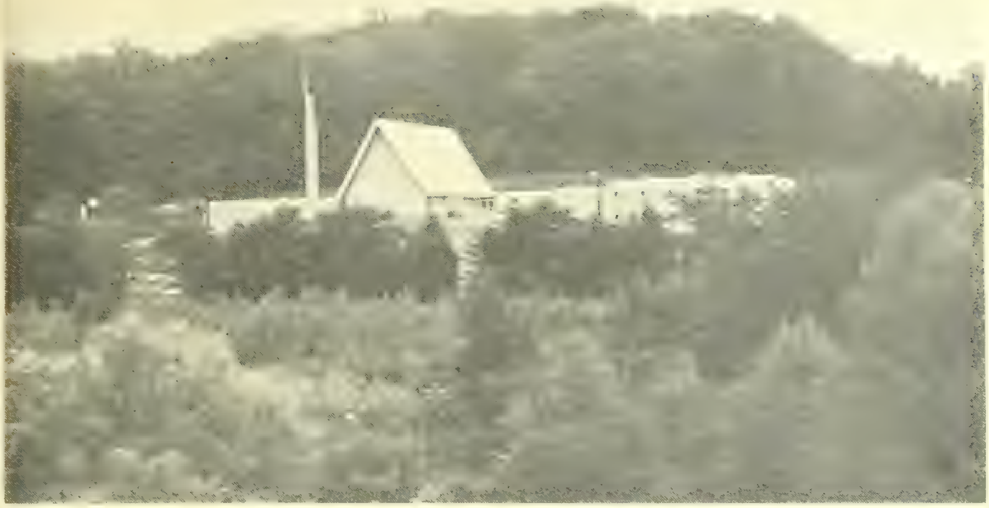


Figure 9.—Mormon church as seen from Capital Beltway, I-495.

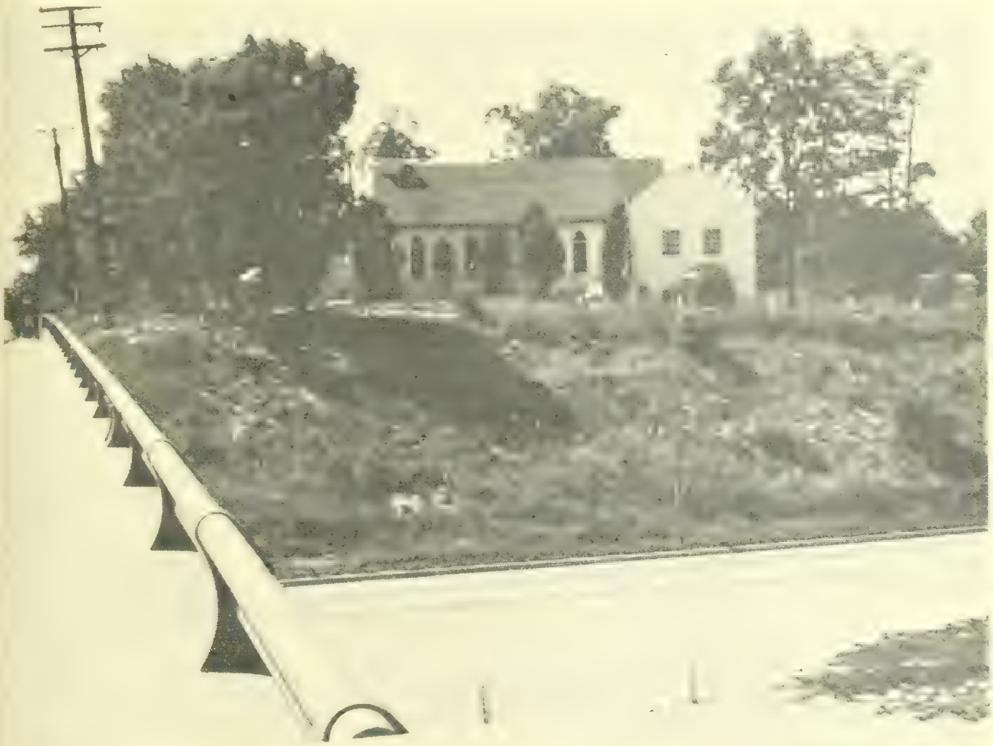


Figure 10.—Ebenezer Methodist Church at intersection of I-495 and U.S.-50.

change opportunities to good advantage. "Only minutes away by the Beltway" and "Exit 30, Capital Beltway" are typical notices that churches use to indicate how quickly and easily they can be found. Church spokesmen emphasize the advantages of interchange accessibility in many ways, such as for areawide meetings and for members who wish to continue in the same congregation after a change of residence. Visibility is also used to good advantage by several churches that do not have easy access. Example of churches visible from the Capital Beltway are shown in figures 9 and 10.

Churches need to be accessible to members, but need not necessarily be near the geographic centers of the members' residential areas. Churches, like shopping centers, can expect to attract fewer trips from the *inboard* or central business district side of a metropolitan area than from the *outboard* side (*2*). To learn more about this *inboard-outboard* effect on a church and how the beltway may change it, a rudimentary analysis was performed using a listing of members' residences for the Christ Congregational Church, Silver Spring, Md., the interchange church shown in figure 11. It appears that fewer members may in fact reside on the *inboard* or central business district side of the church than on the so-called *outboard* side, unless a disproportionately high number of local members reside in the *inboard* side. About 4 percent of the members reside in the area approximated by the quadrant to the *inboard* or south side of the local area of the church; about 20 percent reside in the quadrant on the *outboard* or north side of the *local* area. The *local* area is defined here as that portion of Silver Spring, Md., encompassed by the postal-zip-code area in which the church is located. This area extends out from the church an average of about 2 miles and contains about 44 percent of the residences of the members, as shown in figure 12.

Any beltway effects being experienced by the Christ Congregational Church are apparently minor. Only about 15 percent of the church members arrive via the beltway, yet a church spokesman described the effect of the beltway on church activities as very favorable. This congregation, with a building constructed in 1949, is now planning an expansion. The membership is increasing and apparently coming from farther away. Before the beltway opened, members residing outside the *local* area constituted slightly more than 52 percent of the total congregation. Now, about 56 percent of the new members reside outside the *local* area. The increase seems to be coming predominantly from quadrants to the west and north of the local area. As shown in figure 12, the quadrant to the west of the *local* area formerly contained about 26 percent of the members; now about 27 percent of the new members reside in this area through which the beltway was built. The small portion of members being attracted from the east—1.9 percent before 1954, and 2.6 percent since—might be explained in part by the presence of another Congregational Church there.

Summary

The pilot study reported here has achieved at least some success: It has produced enough information to suggest strongly that church activities at interchange areas may, indeed, be an appropriate land use. The belief that churches near highway interchanges might suffer from noise, fumes, or other highway disturbances—one of the reasons for undertaking the pilot study—apparently is, at least partly, mistaken. While there are still some who prefer church sites readily accessible to pedestrians (*1*), a number of church location spokesmen apparently now believe—or have become reconciled to the fact—that the role of the pedestrian is becoming less important. In another study of a suburban church, only the pastor walked to church.

Although the study did not disclose as many problems as were anticipated, it obviously has not demonstrated that no problems

Figure 11.—Christ Congregational Church at intersection of I-495 and U.S.-29.

are encountered when churches locate near interchange areas, not even in the Washington, D.C., area. More information is needed before definite conclusions can be drawn. For example, it is necessary to show whether churches can, as several spokesmen claim, realize the full potential of high value interchange land. Also, the common assumption that church-generated traffic uses excess capacity at off-peak times may need to be tested because of the trend to use church buildings in the Monday-through-Saturday period. In efforts to analyze and guide interchange area development some attention should probably be given to churches and schools. Also, some of the church-highway matters not dealt with in this paper—the adverse effect churches might have on interchange areas; the location of churches over or above highways to minimize highway barrier effects; church structures over highways or in interchange areas that can be adapted to nonchurch purposes—need at least exploratory research efforts soon. It would also be worthwhile to determine the extent to which church parking space can serve effectively as week day commuter parking as is now taking place in some locations.

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Charles A. Ellet, *The Need for Churches in the Year 2,000,* Council of Churches of Greater Washington, Washington, D.C., 1966.

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Alfred Ray Ganther, *A Catholic Parish Group,* Unpublished master's thesis, Catholic University, 1958.

Edward A. Mueller, *An Investigation of the Traffic Generating Characteristics of a Typical Suburban Church,* Unpublished master's thesis, Catholic University, Washington, D.C., 1967.

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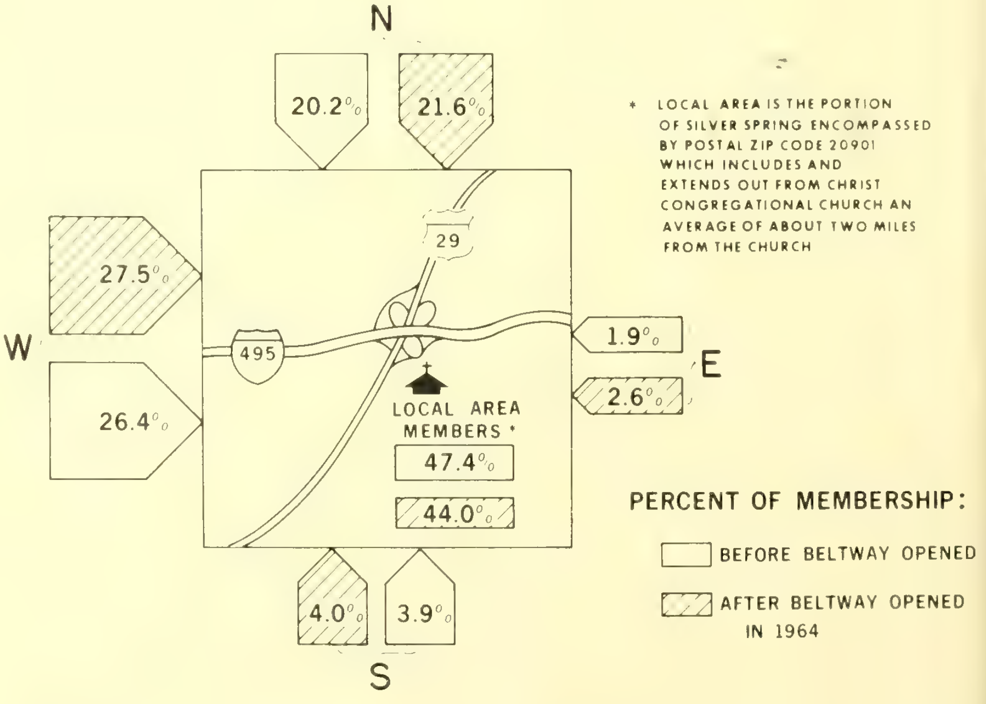
N
20.2%
21.6%
* LOCAL AREA IS THE PORTION OF SILVER SPRING ENCOMPASSED BY POSTAL ZIP CODE 20901 WHICH INCLUDES AND EXTENDS OUT FROM CHRIST CONGREGATIONAL CHURCH AN AVERAGE OF ABOUT TWO MILES FROM THE CHURCH
29
27.5%
W
495
1.9%
E
2.6%
26.4%
LOCAL AREA MEMBERS *
47.4%
44.0%
PERCENT OF MEMBERSHIP:
BEFORE BELTWAY OPENED
AFTER BELTWAY OPENED IN 1964
4.0%
3.9%
S

Figure 12.—Residential locations of members of church at interchange before and after beltway opened.

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Parking as an Element Within the Comprehensive Transportation Planning Process

BY THE
OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by LAWRENCE L. SCHULMAN
Highway Engineer,
Urban Planning Division

Parking exerts a qualifying control on the proposal of any future transportation system, and should be included in its proper perspective in the present comprehensive transportation planning process. Cars destined to an area must be parked before the trip purpose can be fulfilled. If demand for parking exceeds available supply, adjustments must be made among the planned roadway system, the planned transit system, and the planned parking program.

The author evaluates current status of parking analysis and its relation to the comprehensive transportation planning process, discusses the basic relationships behind drivers choice of parking location, and proposes a new procedure for improving the parking analysis and its coordination within the transportation planning process. Discussed in the new procedures are objectives of a parking analysis, detailed description of the procedure, and use of origin-destination survey data for parking demand.

The author details a current parking research project to evaluate some of the possibilities of improving both the coordination of parking within the comprehensive planning process and the general quality of the parking analysis.

Introduction

UNDER the broad guidelines adopted by the Bureau of Public Roads to foster the development of comprehensive, coordinated, and continuing transportation planning in cities of over 50,000 population, significant improvements have been observed in many phases of the process. In contrast, the parking element has remained an unexplored and often an unrelated item within the transportation planning process. Such an observation is particularly distressing when it is realized that parking considerations can influence the proposals of future transportation systems—that is, parking limitations and regulations can influence the level of congestion resulting from the traffic converging within an area; parking facility location can influence the location of roadways and their volume characteristics; and parking availability can influence the determination of transit levels of service.

The parking philosophy is basically simple. In most urban areas, the majority of travel is generated by automobile, and even though the passengers' actual destinations vary, the termination of the automobile vehicle portion of the trip is always some terminal—either curb space (fig. 1), parking lot (fig. 2), garage (fig. 3), or fringe facility (fig. 4). Therefore, just as future trip destinations within an area must be considered in determining future roadway volumes and capacities, provision of parking facilities must also be considered to allow the logical completion of any trip movement. When situations arise in which demands exceed the physical capacity of the area to provide parking, some program of coordination must be developed among the roadway system, the parking program, and the transit system. It is this type of coordinated program that provides the basis for an adequately designed transportation system.

Figure 1.—Curb parking represents a major portion of urban parking supply.

Parking Within the Comprehensive Transportation Process

The conceptual involvement of parking as an element of the transportation planning process is more easily recognized than the methods of introducing it into the process. Should it be a limiting control in the generation of trip ends, a control in the distribution of trip ends, a restraint in traffic assignment, a factor within the modal split, or a combination of all?

Of the several phases in which parking could be introduced, there is one point in the process

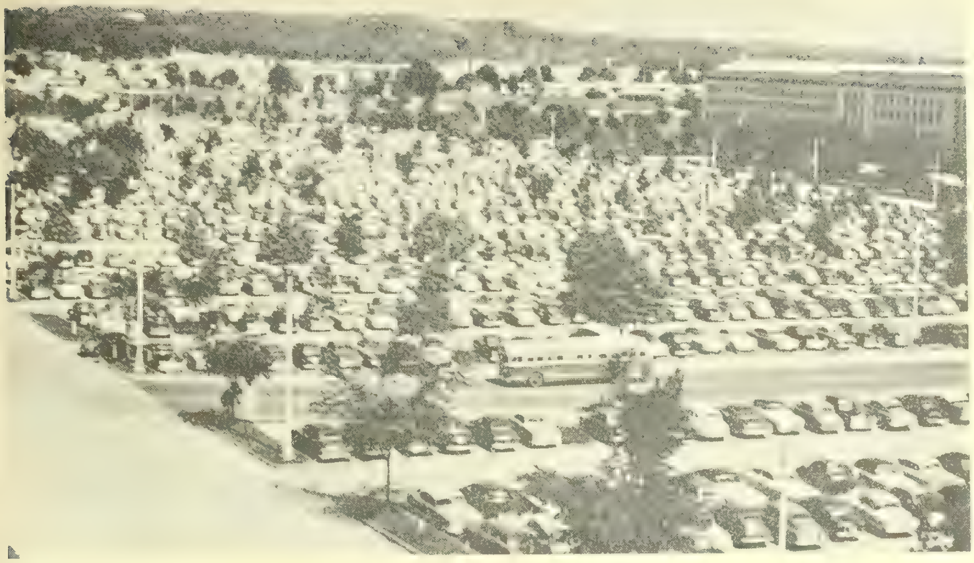


Figure 2.—Parking lots at major new traffic generators frequently use large volumes of urban land.

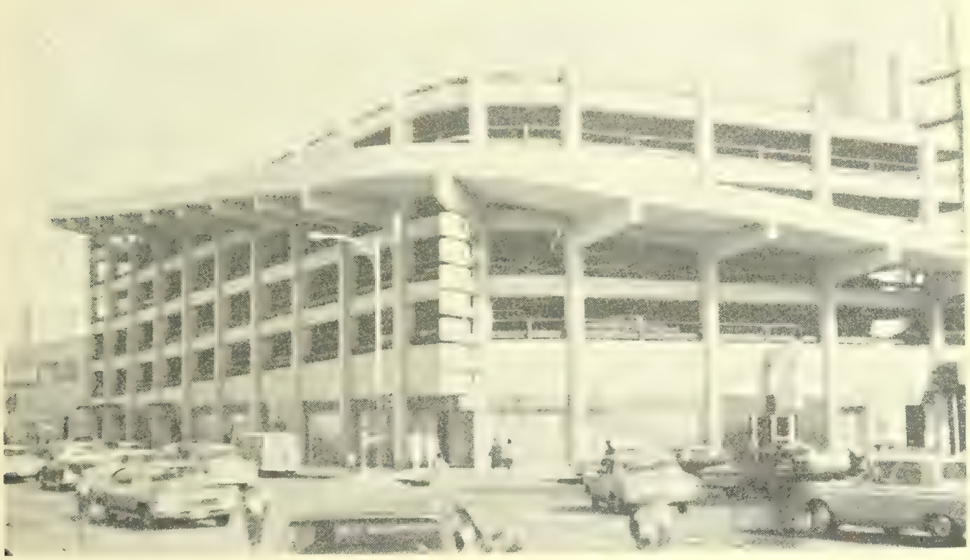


Figure 3.—High urban land costs lead to construction of multistory facilities in many downtown areas.

Figure 4.—Fringe parking requires efficient, low cost transit service to ultimate destinations.

at which adequacy of parking exhibits a controlling condition. That point is immediately after the determination of the zonal auto-driver trip ends. It is at this point that the total number of auto trips destined to each zone is known, and it is at this point that a check should be made to determine whether the zone has the capability of accommodating all the destined trips.

Again, this logic is based on the simple but often forgotten fact that, with few exceptions, every car destined to an area must be parked before the trip purpose can be fulfilled and if the car cannot be parked in an area, it is senseless to design a roadway to bring it there. Therefore, after determining the number of trips that will terminate in a particular area, it must be determined whether the area is able to accommodate all the vehicles requiring parking.

In essence, the parking analysis will involve a process of distributing the demand to the available supply, but the point at which this distribution will enter the phases of the process will depend upon which of the three basic procedures is followed to calculate zonal auto-driver trip ends.

In the first procedure, shown in figure 5, the transit and highway systems are analyzed separately, thereby eliminating the need for a modal-split analysis. This type of analysis would occur in a smaller area where transit plays a minor role in the transportation system. After determining the zonal auto-driver trip ends in the generation phase, they are distributed as parking demand, and the feasibility of accommodating the predicted number of automobiles is evaluated. The results of this evaluation are then analyzed and any obvious inconsistencies are corrected.

In the second procedure, shown in figure 6, the role of transit is considered important, and the two systems—transit and highway—are analyzed together. The trip generation, the trip distribution, and the modal-split phases are all completed before zonal auto-driver trip ends are determined. In this procedure any inconsistencies can be corrected by reevaluating either the trip generation phase, the trip distribution model or the modal-split procedure.

In the third procedure, shown in figure 7, as in the second, the role of transit is considered important. The difference in procedures reflects the arrangement of the internal phases, and in this procedure only the trip generation and the modal-split phases have taken place before the zonal auto-driver trip ends are determined. Here again the auto-driver trip ends are distributed as parking demand, and the resulting parking conditions are examined. If inconsistencies are apparent, then either the modal-split procedures or the trip-generation procedures can be reevaluated.

In each of these procedures, the parking demands have been distributed among the available parking supplies, and any obvious inconsistencies have been assumed to be corrected by changes in one or more of the analytical phases. However, simple changes in municipal policy can cause significant changes in the parking supply and consequently significant changes in the results of the analysis. Therefore, in each of the evaluations, possible changes in supply must also be considered in addition to adjustments of the analytical phases. Consequently, the analysis must be developed so that changes in total person trips, changes in transit service, or changes in parking supply all can be investigated when an attempt is made to balance the desires to travel to an area and the physical capabilities of the area to receive the trips.

The basic assumption in each of these procedures is that zonal auto-driver trip ends are synonymous with zonal parking demand. It is assumed that each trip end represents the demand for a parking space and that the sum of trip ends together with the parking duration represents the total demand for parking in any zone. It is also assumed that the future forecast of zonal auto-driver trip ends represents the future forecast of zonal parking demands.

Suggested Parking Analysis

As envisioned, the two main objectives of the parking analysis should be: (1) To determine the areas in which parking is deficient and the reasons for the deficiencies and (2) to prevent additional deficiencies by evaluating the realism of future trip-end forecasts in light of existing and future parking supplies as reflected in land use forecasts. One of the preliminary requirements for accomplishing these objectives will be the calibration of a simple computer model capable of: (1) Reproducing the process by which automobiles are distributed among available parking supplies, (2) determining the specific areas of parking deficiency and (3) determining the type of parking—short term or long term—required to eliminate the deficiencies.

The procedure contemplated is a simple distribution model, in which the individual block parking demand is distributed among the supplies available within a composite area. As each potential trip approaches the block of destination, a test is made to determine whether there are any satisfactory spaces available in that block. If space is not available, a search is made to determine the next closest block with available space. As each successive block is explored, a check is made to determine whether the parking space is within acceptable walking distance. This procedure is continued until all the demand is satisfied or all the spaces within acceptable walking distances are exhausted. This simple distribution process is shown in figure 8.

The entire distribution is dependent upon the outlined process; however, the degree of complexity increases as the demand is stratified by parking duration and time of arrival; as supply is stratified by time restriction and cost; and as maximum acceptable walking distances (or composite areas) change with duration, costs, and parking congestion. These stratifications are reflective of theoretical considerations discussed later, and should allow a realistic approximation of the distribution of parked vehicles within the study area.

To facilitate a detailed explanation, the parking analysis can be divided into four phases—data collection, data analysis, model calibration, and final report.

Data collection

The data collection phase is of primary importance as it not only supplies information needed to determine and analyze existing parking characteristics of the area, but also the data needed for calibration of the distribution model. The information usually required is the actual location of parking, location of demand, time of arrival, duration of parking, and detailed characteristics of the supply (time restrictions, costs, location, etc.).

There are several possible methods available to collect these data. Probably the best method is the one described under "Current Parking Analysis." Historically, this method has been handled in great detail and the resultant data provides all the information necessary.

Another possible method of accumulating the parking data is to use the information collected during the origin and destination (O–D) data collection survey. Much of the required parking data can be determined directly from the trip cards (trip purpose, demand destination, arrival time, and type of parking), and with few exceptions, the rest can be calculated with little effort; that is, parking duration equals the difference between arrival time of the trip to the central business district (CBD) and the starting time of the trip from the CBD. The only information that is not readily available is the actual location of parking, but this could be obtained by revising standard interview forms to include the question "where did you park your car?" or by conducting a usage study as described under "Current Parking Analysis." The characteristics of the parking supply could be obtained by including a detailed study within the land use data collection procedures.

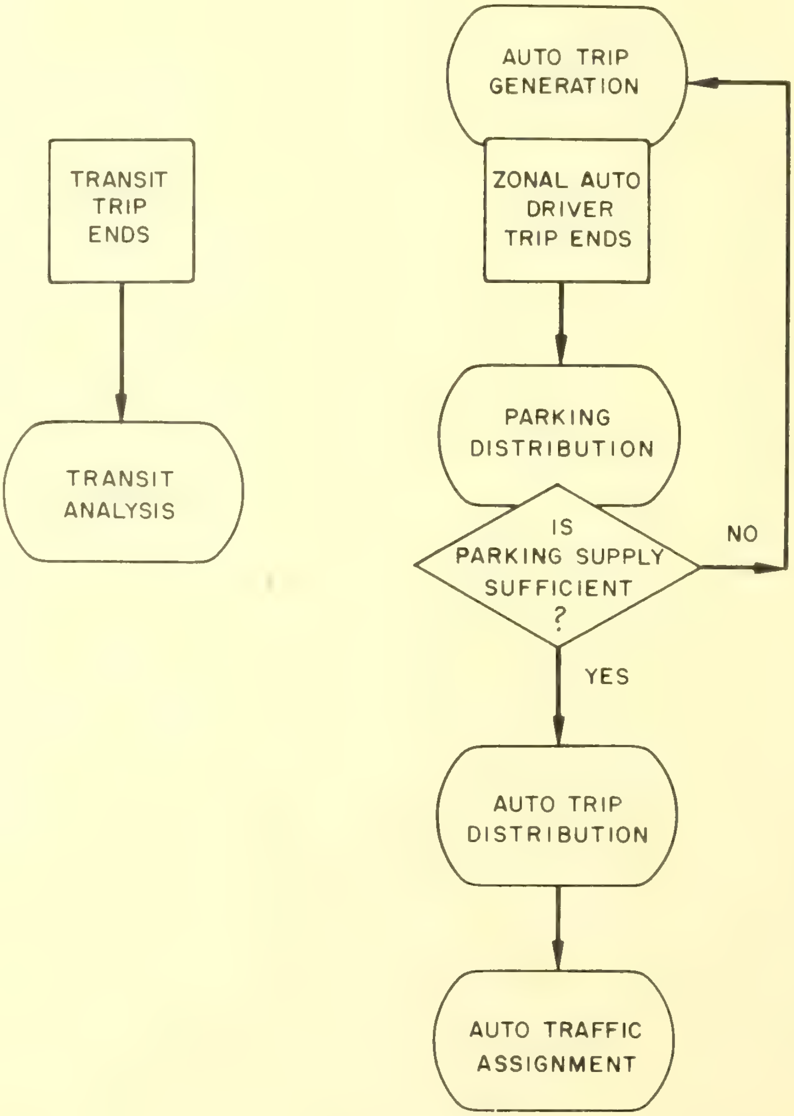


Figure 5.—Phase sequence of planning process without modal split.

Although this method of data collection would truly coordinate the parking analysis within the existing transportation process, the method has not been sufficiently explored, and more research is necessary to determine whether the data obtained would be sufficiently accurate for a detailed parking analysis.

Several comments can be made concerning the advantages and disadvantages of using either method. The first disadvantage of O–D data, and probably the greatest objection to its use, is the limited number of observations available for analysis. This limitation is caused by the obvious anomaly of the sample selection. The selection of O–D data is based on the non-CBD end of the trip. In other words, a 10 percent home interview is a 10 percent sample of the dwelling units and, consequently, a sample of 10 percent of all trips made, including those to the CBD. This type of sample may be sufficient for the more general aspects of the transportation study, but there is some question as to its sufficiency for the detailed stratification necessary in the CBD parking analysis. By comparison, the data collected from a special parking interview can represent as much as 100 percent of the CBD trips. The actual percent interviewed would depend on the size of the area and the staff available.

A second disadvantage in the use of O–D data is the inherent underreporting of the nonhome based trips. The consequence of this shortcoming is manifested in the smaller number of intra-CBD trips reported using O–D data as compared with the intra-CBD trips tabulated from a parking study. There has been limited research in this area, but all the results have indicated an underreporting in the O–D data.

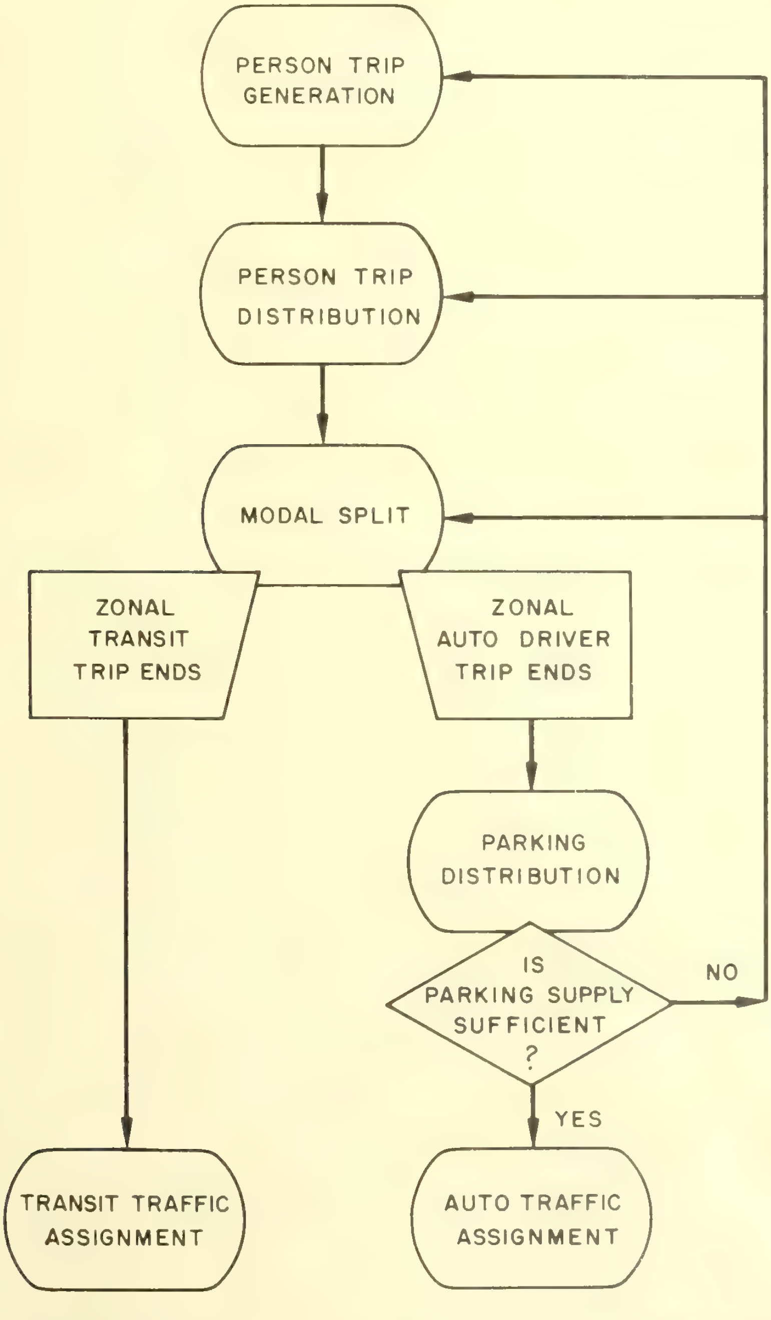


Figure 6.—Phase sequence of planning process using a trip interchange modal split.

A third disadvantage is the handling of the external–internal trips. Usually the information gathered during the external interview is much more limited than that in the home interview portion and consequently, the data needed for the parking analysis is not available. Therefore, some method of approximation becomes necessary in handling these trips. This is not a problem in the areas having a small percentage of external–internal trips; but the larger this percentage becomes, the greater the magnitude of the problem. In the parking interview this problem does not occur as all interviews are conducted at the parking end and the data available are the same regardless of the origin of the trip.

A fourth disadvantage is the lack of enforcement data. During the usage study of the conventional parking analysis, information is collected on the numbers of vehicles double parked, parked in illegal spaces, or parked for greater than the allowable duration. Although such information usually is of little importance, it can be indicative of the degree of congestion in the area. Excessive double parking for example may indicate the need for more short-time parking in an area. Such information may also indicate an improper balance of facility types. For example, excessive parking at greater than the permissible durations may indicate the need for less short-duration spaces and more long-duration spaces.

On the other hand, there are several distinct advantages in using O–D data. The most obvious is the ease of data collection and related financial savings. As pointed out, most of the information needed is collected in the O–D survey and therefore parking data can be obtained by a computerized extrapolation instead of by a second lengthy interview, which would require the restaffing of interviewers, a second session of training, additional editing and coding, etc. Elimination of these repetitious steps would obviously be a financial savings.

A second advantage of using O–D data, is that the data is available for a 24-hour period. This allows the flexibility of permitting an analysis of the data within any desired time period. By comparison, the analysis based on the parking interview is confined to a previously determined time span and cannot be changed once the data are collected.

A third advantage of using O–D data is that all the information required is available for each observed vehicle; that is, there are no observations of parked vehicles with unknown data—a situation that frequently occurs in the parking study, especially if interviewing begins late in the day. For instance, if interviewing begins at 9 a.m., an auto-driver parked at 8 a.m. and remaining parked all day would not be interviewed and the parking characteristics could not be obtained even though the vehicle is observed in the usage study. This cannot occur in the O–D data as the trip, if recorded at all, will provide the required information.

Data analysis

Regardless of the procedure employed to collect the data, the analysis and the distribution process are basically the same. The data collection phase merely supplies the required data and is independent of the subsequent analysis. This is not true of the next two phases; data analysis and parking distribution are strongly interrelated. The type and the detail of the data analysis is dependent on the level of detail required as input in the parking distribution—the more complex the distribution, the more detailed the analysis.

As one of the initial tasks in the analysis will be the calibration of a computer model, the characteristics of parking must be reduced to a series of simple but logical mathematical relationships. In addition, other pertinent factors must be determined to facilitate a check on the validity of the model distribution. The detailed analysis is difficult to explain at this point because the details depend on the specifics of the model, but in general the following relations are required:

- Average parking durations for different purposes and different arrival times.
- Acceptable walking distances for different durations under various cost structures and various levels of congestion.
- Arrival of trips by purpose distributed by hour of arrival.

A more detailed discussion will be possible once the distribution model is fully defined.

Model calibration

Before any hypothetical model can be used, it must be calibrated based on the existing data. The method of calibration must allow the distribution of the entire parking demand at farther than acceptable walking distances, and provide a check to determine how closely the model distribution compares with the actual parking distribution. Based on this comparison, adjustments can be made in the theoretical model to better reproduce the base year conditions.

After calibration the model is ready to be used to evaluate either existing or future parking conditions, or to evaluate the effects of changes in either cost structure or parking supply within the CBD. As a tool within the transportation planning process, it can also be used to evaluate the effects of changes in level of transit service, the effect of introducing a fringe parking program, or the effect of major land use changes.

Final report

The parking study report should provide a clear-cut analysis of the parking situation as it exists in the base year and as it will exist in the horizon year based on a forecast of future trips. The report should include a breakdown of the type and general location of parking; but more important, it should show the effects of changing the existing parking supply characteristics, costs, or durations; of increasing or decreasing the number of spaces available; of changing the amount or level of transit service; or of introducing fringe parking. In other words, the report should present a series of comparative analyses based on the different alternate parking plans possible within the transportation system.

Basic Relations

The parking phenomena is based on the laws of supply and demand, where supply is the total number of spaces available within a specified area and demand is the desire to park at a specified location. However, unlike a true supply and demand situation, this basic relationship is confounded by a third variable, usage, where usage reflects the actual location of parking.

At first it might seem that demand and usage are identical, but closer observation shows that this is the exception rather than the rule. Demand is the desire to park based solely on the location of the trip destination, whereas usage reflects the desire to park close to the destination, but within the limitations imposed by available supply, and the desire to park at a reasonable cost. Thus, demand is a constant, reflecting only the desire to park at the trip destination location, whereas usage is a variable that depends on the conditions

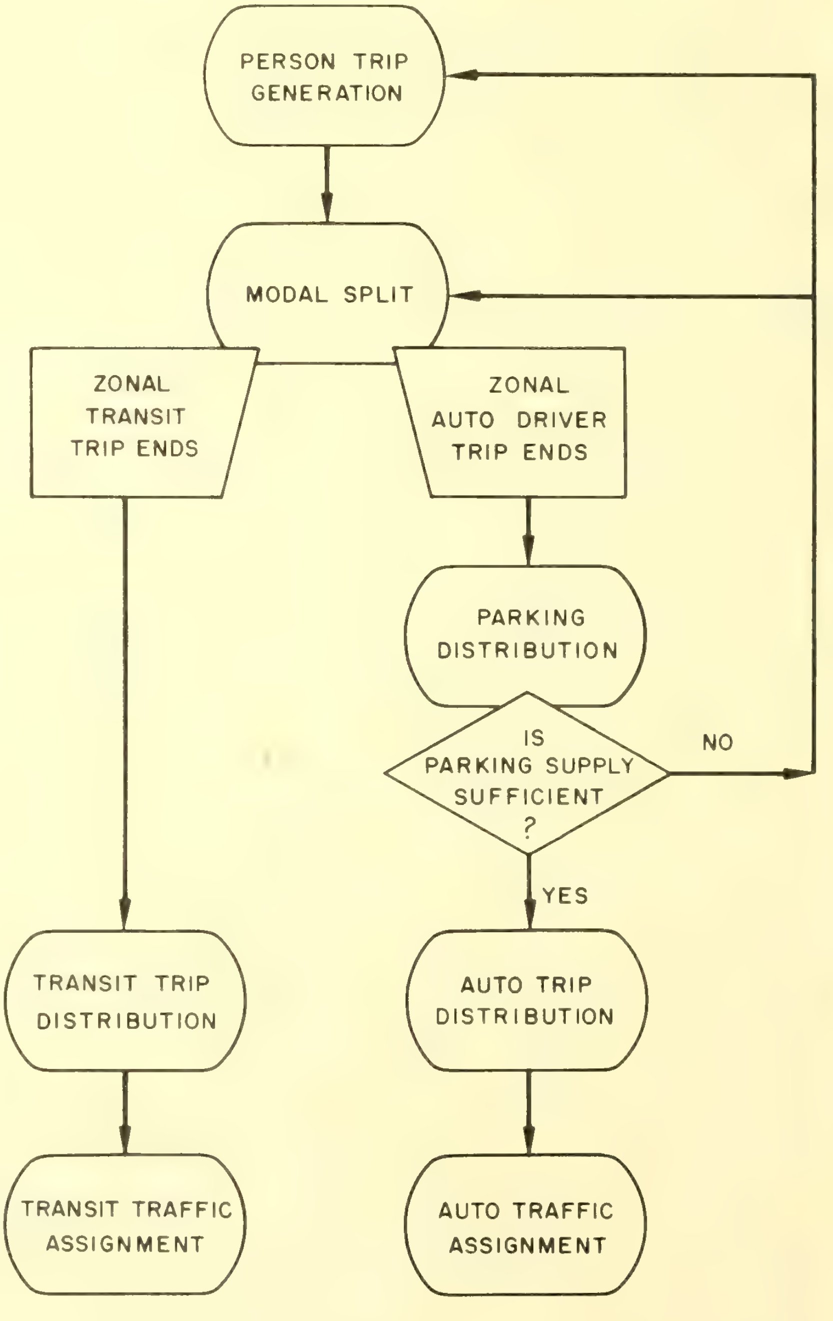


Figure 7.—Phase sequence of planning process using a trip-end modal split.

NEW PUBLICATIONS

The Bureau of Public Roads has recently published three documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Highway Statistics, 1966

Highway Statistics, 1966, is the 22d issue of the annual compilation of statistical and analytical tabular matter pertaining to Federal-aid for highways. This 186-page publication ($1.25 a copy) presents information, primarily in tabular form, on motor fuel, motor vehicles, driver licensing, highway-user taxation, State and local highway financing, road and street mileage, and Federal-aid for highways.

Capacity Analysis Techniques for Design of Signalized Intersections

Capacity Analysis Techniques for Design of Signalized Intersections—Reprinted from August and October 1967 issues of PUBLIC ROADS, A Journal of Highway Research (45 cents a copy)—are presented for the graphic solution of capacity problems related to signalized intersections. The procedures are based on a set of charts consisting of 20 nomographs, which were devised by the author, Jack E. Leisch, in 1950, to simplify the computational procedures of the 1950 Highway Capacity Manual. They were presented in PUBLIC ROADS, A Journal of Highway Research, in 1951, and were acclaimed by those concerned with intersection design. Publication of the 1965 Highway Capacity Manual has provided a revised and comprehensive basis for capacity computations, and the author has again filled the need for a graphic procedure incorporating current knowledge. The original charts have been updated and new charts prepared to cover capacity procedures for which calculations previously required extensive application of judgment. The information presented provides a graphic procedure for the capacity analysis of most signalized street and highway intersections. Full discussion of the principles and procedures in the application of the charts, in addition to sample problems, have been included.

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds

The 1967 issue of *Highway Research and Development Studies Using Federal-Aid Research and Planning Funds* may be purchased for $1.00 a copy. The studies listed are those approved in the Office of Research and Development, Bureau of Public Roads, FHWA, for fiscal year 1968, and calendar year 1967, as of July 1, 1967.

The information has been grouped by the seven major technical goals of the National Program of Research and Development in Highway Transportation. These major technical goals are:

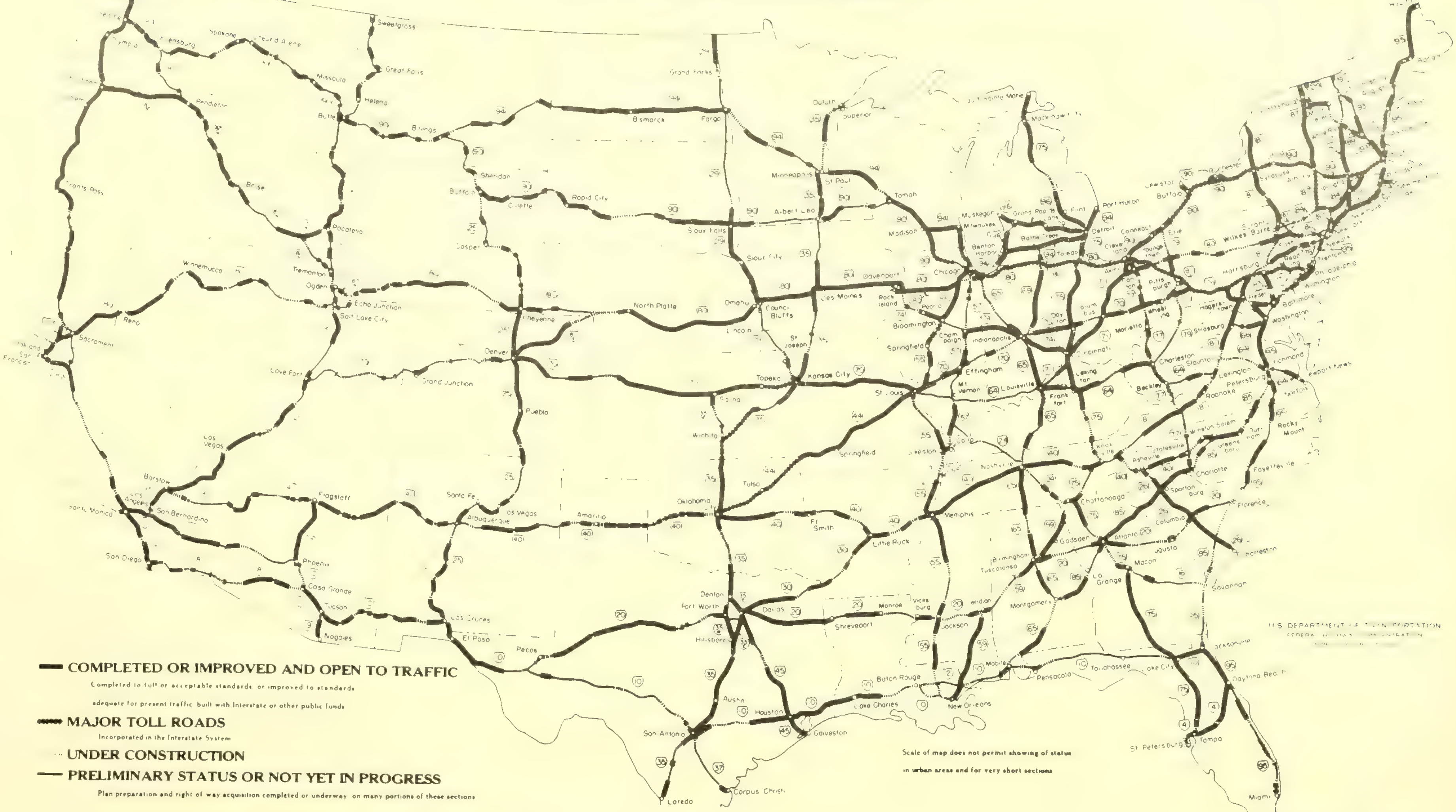
Definition of underlying requirements for highway transport,
Analytic definition of complex traffic movements,
Analysis of essential components of highway transport,
Development of methods for reliable forecasting of demand for highway transport,
Development of methods for increased capacity, control, and safety in traffic movement,
Development of techniques for more precise structural design and incorporation of new materials and structural concepts,
Development and application to new technology to location, design, construction, and maintenance processes.

An eighth grouping includes miscellaneous projects of local, regional, or national importance. Data are also presented on the objective of each study, the conducting agency, and the funding for each study.

This year's edition contains a new feature—a section listing available reports on Federal-aid highway research studies. In addition, a newly revised and amplified foreword briefly explains the makeup and operation of the Federal-aid highway research and development program, and describes how the content of the publication is organized. The expanded foreword is intended not only for Federal Highway Administration personnel, but also to guide those outside the Government who are interested in the program and may desire to participate in it.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF DECEMBER 31, 1967



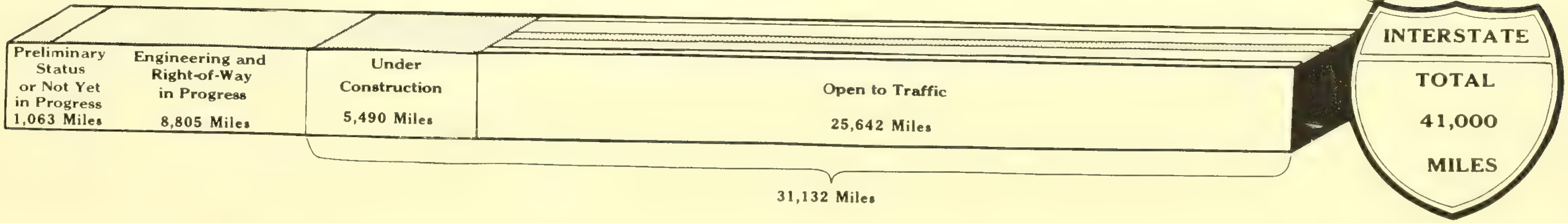
━ COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards or improved to standards adequate for present traffic built with Interstate or other public funds

▬ MAJOR TOLL ROADS
Incorporated in the Interstate System

⋯ UNDER CONSTRUCTION

— PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right of way acquisition completed or underway on many portions of these sections

Scale of map does not permit showing of status in urban areas and for very short sections



U.S. GOVERNMENT PRINTING OFFICE: 1968

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–34 are available upon request addressed to Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. 20591.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

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  1963, 35 cents. 1964, 35 cents. 1965, 40 cents. 1966, 75 cents. 1966 supplement, 25 cents.
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The Bridge to Your Success—Opportunities for Young Engineers in the Bureau of Public Roads (1967). 30 cents.
Calibrating and Testing a Gravity Model for Any Size Urban Area (1965). $1.00.
Capacity Analysis Techniques for Design of Signalized Intersections (Reprint of August and October 1967 issues of PUBLIC ROADS, a Journal of Highway Research). 45 cents.
Construction Safety Requirements, Federal Highway Projects (1966). 55 cents.
Corrugated Metal Pipe Culverts (1966). 25 cents.
Creating, Organizing, & Reporting Highway Needs Studies (Highway Planning Technical Report No. 1) (1963). 15 cents.
Federal-Aid Highway Map (42 x 65 inches) (1965). $1.50.
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Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.
Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (1967). $1.00.
Highway Statistics (published annually since 1945):
  1965, $1.00, 1966, $1.25.
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Highway Statistics, Summary to 1965 (1967). $1.25.
Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.
Highways to Beauty (1966). 20 cents.
Highways and Economic and Social Changes (1964). $1.25.
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Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
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Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.
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Traffic Safety Services, Directory of National Organizations (1963). 15 cents.
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June 1968

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

Public Roads

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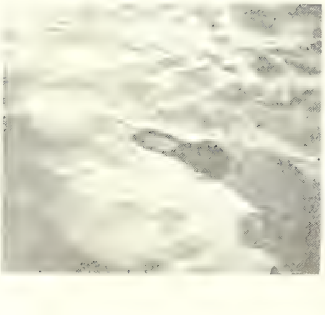

COVER

Satellite photograph obtained at an altitude of 110 nautical miles from Gemini V (Courtesy NASA). Applications of remote-photographic techniques to highway problems are discussed in the first article.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 16, 1966.

Contents of this publication may be reprinted. Mention of source is requested.

Remote Sensing Applications to Highway Engineering

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

Reported by[1] **HAROLD T. RIB**
Highway Research Engineer

In recent years, the availability of new remote sensors—radar, infrared, multichannel—has initiated a new era in the application of interpretation techniques in highway engineering. The data provided by the many types of sensors furnish information previously unobtainable, or available only at great expense. In this paper, the areas of possible applications of remote sensors in the highway field are discussed. Several examples are included to demonstrate the value of remote sensors in various aspects of highway engineering. Also included is a discussion of the Public Roads research program in remote sensing.

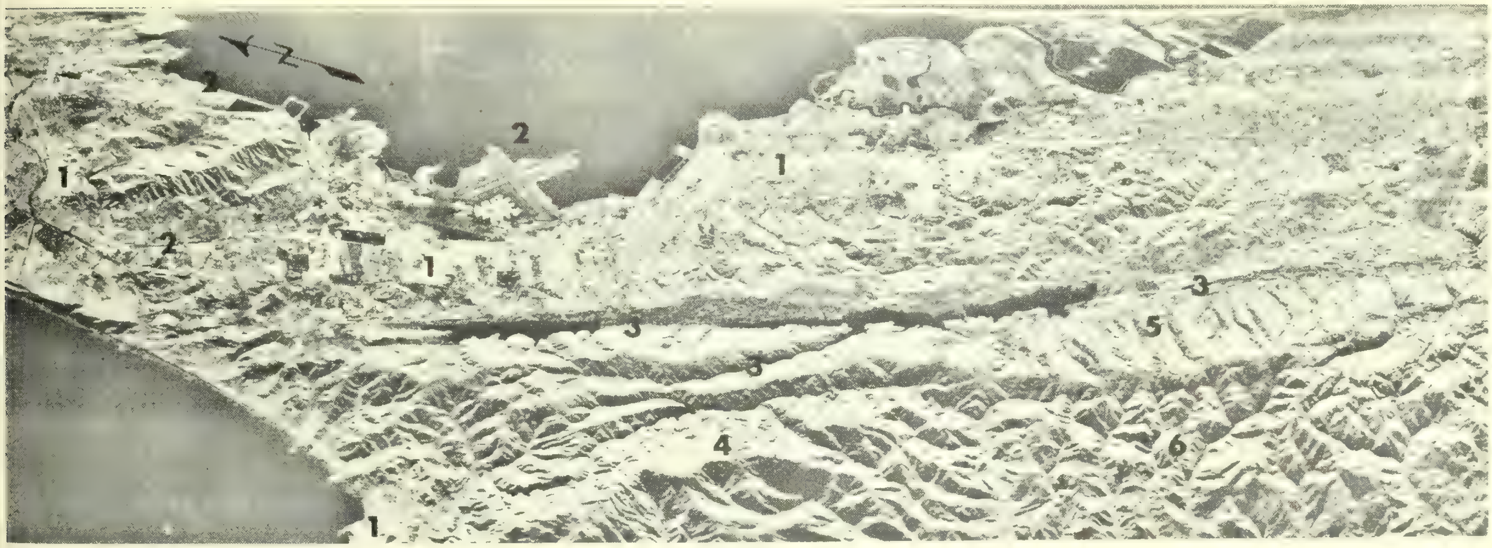

Figure 1.—Radar image of portion of San Francisco peninsula illustrating regional view obtained by side-looking radar and its ability to distinguish numerous features important to highway planning. Numbered items are discussed in text.[2]

Introduction

THE HIGHWAY field has entered a new era in the application of aerial reconnaissance techniques, now referred to as aerial remote sensing. This era has been ushered in by the availability of new sensors to civilian organizations and the potential to obtain broad regional coverage from satellite platforms. The new sensors include infrared, radar, and multichannel and multiband instruments. In addition, the improvement and availability of aerial film types, including natural color, color infrared, and color negative, and the development and use of special film-filter combinations for image enhancement, have aided in promoting this new era. Preliminary studies in the application of the newer sensors and techniques have demonstrated that, individually or in combination, these newer systems procure information previously unobtainable, or available only at great expense.

To describe the applications of these newer aerial remote sensors to highway engineering, six major stages of highway engineering are differentiated in this article: Highway planning surveys; condition and inventory surveys; traffic surveys; highway location surveys; construction surveys; and maintenance surveys. For each of these stages, the major factors or data required are indicated; the potential applications of the newer sensors are discussed; and research projects in progress, with some examples demonstrating the applications, are included. Finally, there is a description of the Bureau of Public Roads' research program in remote sensing as well as a discussion of research in progress.

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[1] Presented at the Fifth Remote Sensing Symposium, University of Michigan, April 16-18, 1968.

[2] Photograph courtesy National Aeronautics and Space Administration.

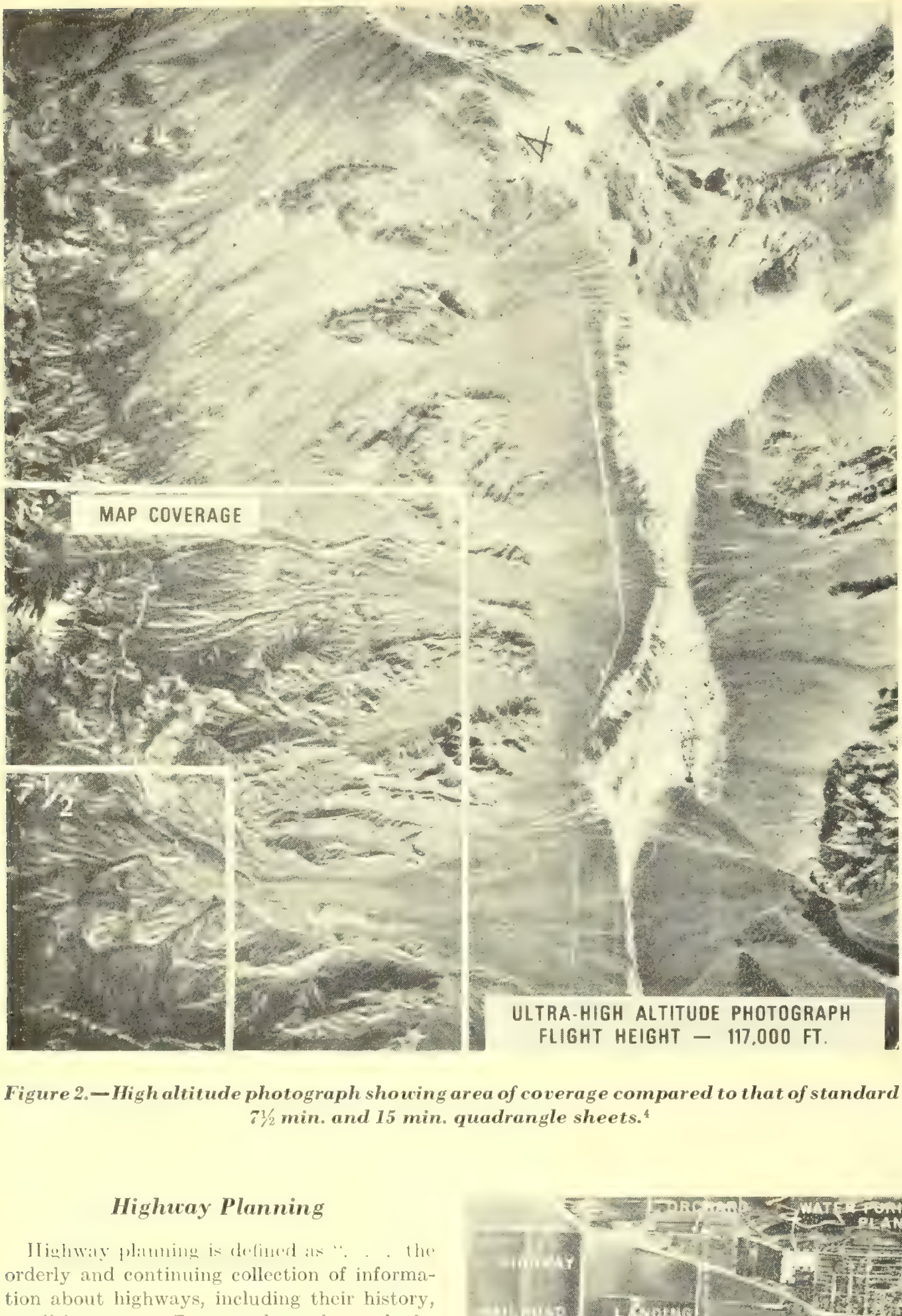

Figure 2.—High altitude photograph showing area of coverage compared to that of standard 7½ min. and 15 min. quadrangle sheets.[4]

Highway Planning

Highway planning is defined as ". . . the orderly and continuing collection of information about highways, including their history, condition, use, effects and needs, and the analysis of these data for the efficient and economic development of the highways systems" (*1*).[3] Proper highway planning must be comprehensive and coordinated. It should be part of an overall master plan for area or regional development, and should be coordinated with that of other agencies and organizations in the development of transportation facilities. The importance of these items is evident in recent planning trends. Comprehensive planning in larger communities required by law. Congress passed a bill quiring all urban areas containing more th 50,000 persons to have a comprehensi transportation plan to receive Federal a Coordinated planning is illustrated by t team concept in planning that is underw in Baltimore, Md. In this program, a team architects, city planners, sociologists, eco mists, and others are working with highw engineers on routing a section of Intersta highway through the city. The planners this highway corridor will also consider oth community needs, including replaceme housing, parking, recreational, commerci and other community facilities.[5]

Another important factor in planning that some studies contemplated today a of broad regional extent. Regional studi may include extensive transportation corrido that may even extend outside the boundari of a single State. An example of this is t study of transportation systems in the nort east corridor extending from Washington, D.C to Boston, Mass.

Remote sensing offers unique advantag in this area of regional planning and coord nation. Radar or satellite photography provid large regional coverage in which sufficie details for general planning purposes a obtained. For example, the side looking rad image in figure 1 shows the following importa features in highway planning: Populatio centers—urban, rural, and other land u patterns (1); transportation systems—hig ways, railroads, airports (2); geologic stru ture and lithology—faults (3), difference rock types (4), (5), and (6). Drainage pattern and many other pertinent details also a evident. Similar regional factors can b evaluated from satellite photography (se cover photograph) and from very hig altitude photography, as shown in figure The ability to judge the influence of the man factors, which is possible through these form of remote sensing, makes the technique very valuable planning tool. Many of thes

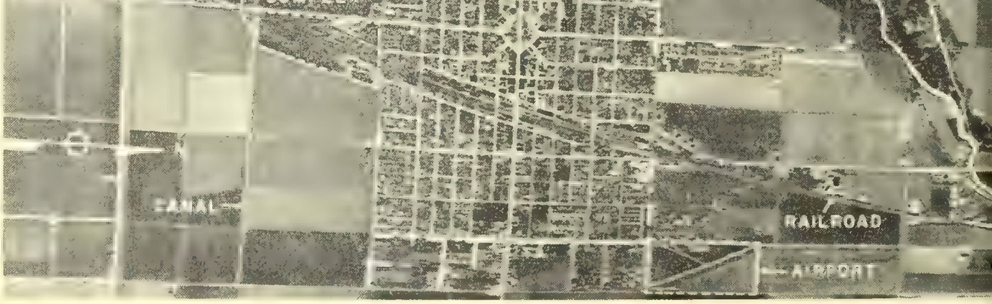

Figure 3.—Nighttime infrared imagery showing type of information obtainable fro views of populated areas.[6]

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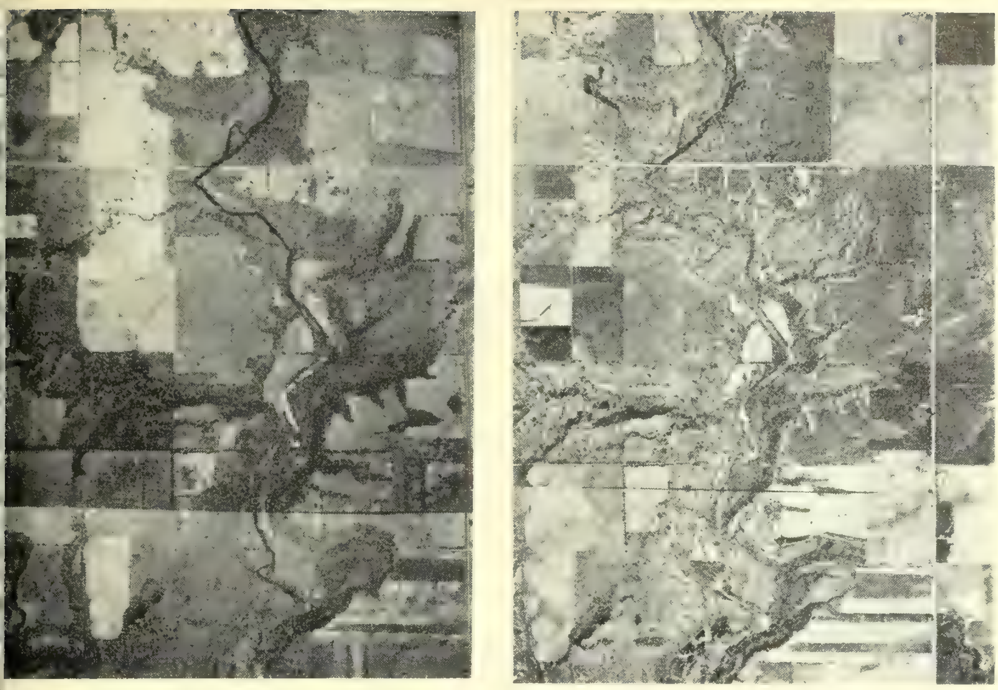
[3] Italic numbers in parentheses identify the references listed on page 36.

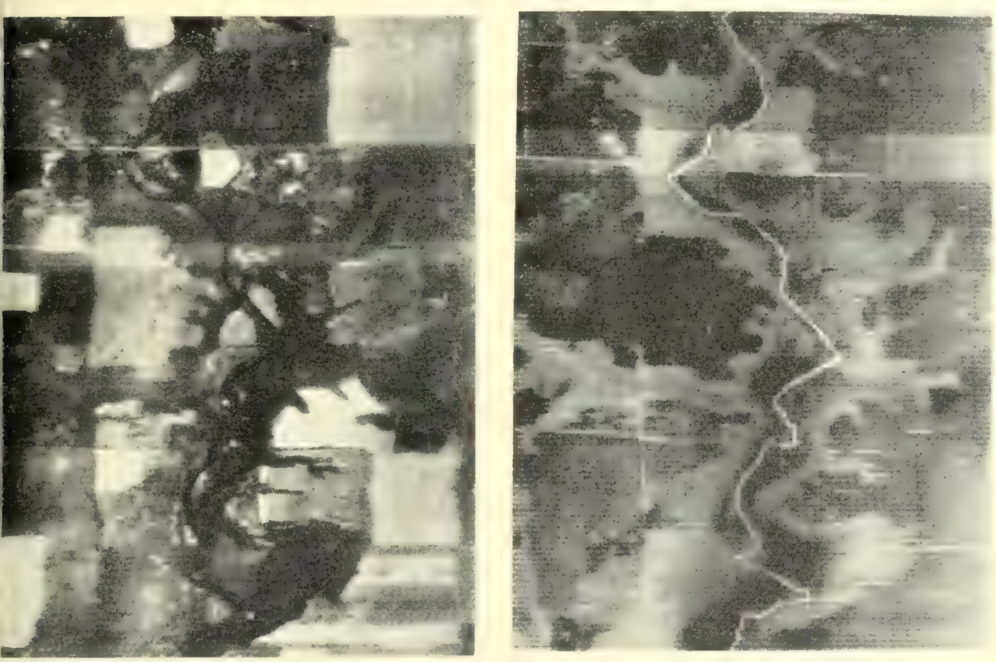
[4] Photograph courtesy U.S. Geological Survey.

[5] From remarks by L. K. Bridwell before the Rubber Manufacturers Association (see acknowledgments).

[6] Photograph courtesy HRB Singer Inc.



Black and white infrared. *Black and white aerial photography.*



Infrared daytime. *Infrared nighttime.*

igure 4.—Drainage patterns evident on various aerial views of photography and infrared imagery.

actors, and much of the interplay between hem, cannot be evaluated rapidly or easily y other methods.

Radar offers a particular advantage in the ighway planning stage because coverage of arge areas can be obtained rapidly, regardless f weather or illumination conditions—day or ight, cloudy, raining, etc. Moreover, the adar mosaic has a fairly continuous tonal attern over the complete area, a character-stic that makes it easier to correlate various tems throughout the area. The ability to correlate tonal patterns over a photographic mosaic of an equivalent area is not always possible. Often the tonal differences present are not due to differences in objects alone. They can be due to differences of exposure and development between the individual photographs used to prepare the photographic mosaic.

Other items pertinent to highway planning include condition and inventory surveys and traffic surveys. These items are discussed as separate stages in this paper. In the final analysis, however, data determined from these surveys would be included in the long-range planning of a State highway system.

Condition and Inventory Surveys

Condition and inventory surveys are performed to compile statistics on: (1) mileage of the functional classifications of highways; (2) kind and number of structures; (3) road surface types, widths, and condition; (4) information on land uses bordering the highway; and (5) other data. Periodic evaluation and updating of highway conditions are required for the proper planning of a State highway system. Some of the detailed data needed in the inventory, such as sight distances, riding quality, superelevations, and clearances, can be determined only by field investigations. However, a good deal of the necessary data can be obtained by aerial remote sensing techniques.

The functional classification and mileage of the different surface types of roads—concrete, bituminous, gravel—are important items in inventory surveys. Multisensor coverage, or coverage in various spectral regions of the electromagnetic spectrum, offers a unique method to distinguish the road surfaces. This ability is demonstrated in table 1, in which tonal signatures are listed for various road surfaces in different regions of the spectrum. The values shown are average values, based on the analysis of photographic and imagery data collected over a 13-month period at an Indiana test site. Significant trends, as seen in the table are: (1) highway surfaces can be determined from multisensor coverage, and (2) at least three spectral regions have to be sampled for identification, that is, visible, photographic infrared, infrared. Figure 3 is a nighttime infrared image illustrating road detail and other information valuable to highway planning.

The results obtained indicate a great potential for the remote sensing approach in identifying various types of road surfaces. It is anticipated that with present day sensors, such as the University of Michigan's multichannel sensor, and present day electronic and computer equipment, a method can be developed to automatically map and classify highway systems. This is one of the items being investigated in Public Roads' remote sensing research program.

Remote sensing techniques can also be used to obtain information on numerous other items pertinent to condition and inventory surveys. Information such as road widths, road conditions, drainage conditions, kinds of structures, land use, rail crossings, etc., can readily be ascertained from natural or infrared color aerial photography. Many of the details can be determined on small scale photography (1:20,000–1:60,000), whereas, some would require medium scale photography (1:8,000–1:20,000).

The natural appearance of objects on natural color photography makes them easier to identify. On color infrared photography, road boundaries, road conditions (patching, break-up, etc.), and drainage features (drainage channels, seepage areas, sediment and pollution) are distinct. Studies by Rib (*2*) and Mintzer [7] have demonstrated that finer details and smaller objects can be more positively identified on color photography than on equivalent black-and-white photography at the same scale.

Considering the scope and amount of information needed for condition and inventory surveys, it would appear that the maximum amount of information could be obtained from aerial remote sensing flights simultaneously procuring multisensor imagery and aerial color photography.

Table 1.—Multispectral tonal signatures for typical highway surfaces

Highway surface	Tonal signatures [1]			
	Panchromatic (No. 12 filter)	Black-and-white infrared (No. 12 filter)	Infrared imagery (8–14μ)	
			Daytime	Nighttime
Concrete	Light	Medium	Medium light	Light.
Asphalt:				
Fresh	Dark	Dark	Light	Light.
Weathered	Medium light	Medium	Medium	Light.
Gravel	Light	Light	Medium	Medium light.
Cinders	Dark	Dark	Light	Medium light.

[1] Five tones distinguished—light, medium light, medium, medium dark, and dark.

Traffic Surveys

Information on traffic is an important factor in planning new highways and in evaluating the capabilities and need for improvement of existing highways. Factors required in traffic surveys include volume counts, classification (size, weight, vehicle type), speed, spacing, origin and destination data, as well as evaluation of regional land use for possible generation of traffic.

Many of the factors, including traffic density, speed, headway, occupancy, variation of traffic density with time, areas of congestion, and bottlenecks, have been evaluated from analysis of black-and-white photography. However, these have been evaluated only for specific cases because of limitations of the aerial photography. Weather and illumination conditions can effect the ability to gather data during critical periods of traffic flow such as the evening rush hour. In addition, the normal aerial photographic flights for traffic surveys are limited to the instantaneous area coverage obtained on each exposure. This limitation can be critical when considering the needs for accomplishing large regional corridor studies.

A potential solution for obtaining instantaneous, as well as periodic, coverage of large regional areas is the use of satellites. Photographs of large regional areas would show conditions that exist within major population centers at a given instant, or between major population centers within a fairly short period. Where weather or illumination conditions restrict procurement of photographic coverage, radar sensors might be used.

Some major limitations also exist in the use of satellites. These include: (1) resolution capabilities of sensors to distinguish small objects such as cars, (2) need to place the satellites in particular orbits to cover the regions of interest, and (3) cost of utilizing this technique. Nevertheless, the use of satellites needs to be investigated as it may prove to be the only practical way of obtaining simultaneous information of large regional traffic patterns.

Remote sensors also offer advantages for obtaining detailed traffic information over limited areas. Both infrared and radar sensors offer opportunities to obtain critical information at any time of day, and in the case of radar, in almost any type of weather condition in which an aircraft can fly. The use of lasers and television cameras are also fruitful areas to be investigated. The Bureau of Public Roads has contracted with private industry to determine the feasibility of the development of an area traffic surveillance and control system. This project is discussed in more detail in the section on Public Roads research.

Highway Location Surveys

Highway location surveys usually consist of four substages: (1) reconnaissances survey of an area to determine feasible routes; (2) reconnaissance survey of route alternatives to select a route; (3) preliminary survey of the selected route; and (4) location survey staking of the designed location on the ground. The type of information needed in these substages varies from small scale regional coverage in the first substage, in which general parameters are evaluated, to limited large scale coverage in the concluding substages in which numerous details such as location of property boundaries, selection of control points for targeting, etc., are needed. These extreme requirements have necessitated the use of two or more scales of photography for accomplishing highway location and design.

Some of the major items obtained by surveys and evaluated in highway location and design include: (1) land use—severances, costs, access, cultural items (archeological sites, cemeteries, scenic viewpoints); (2) geology—type of rock, topography, presence of faults and fracture zones; (3) soils and soil conditions—presence of poor soils, erodible soils, swamps, landslides, high water table, springs, and seepage zones; (4) drainage—drainage pattern, structure needs, watershed areas, flooding conditions; (5) traffic—control points, origin–destination, volume; (6) construction material—consolidated, unconsolidated, quality, quantity, proximity; and (7) plans for boring programs and field surveys. Aerial reconnaissance techniques received maximum use in the highways location survey stage in the past (*3*) and, similarly, is the stage where maximum application of the newer remote sensing techniques can b applied.

The value of remote sensors, used both individually and in combinations, has been reported by numerous investigators. The use of radar for area reconnaissance, specifically for determining geologic features and land use patterns, has been demonstrated by Barr and Miles,[8] Dellwig,[9] Morrain and Simonett (*4*) and Sheridan.[10] The ability to observe large areas under uniformly illuminated conditions has enabled researchers to determine regional structural and faulting patterns, as well as to differentiate certain rock formations that were neither distinct nor interpretable by other methods. Analysis by satellite photography also has good potential application in this substage.

For the reconnaissance survey of route alternatives and for subsequent substages, multichannel, multisensor, multiband and color photography have distinct applications. Comparisons and evaluations of natural and infrared color photography with standard black-and-white photography have demonstrated the superiority of color. For determining soils and soil conditions, it has been indicated (*2*) that natural color was the most useful single film type. The natural color tones of objects made them easier to identify. Also, it was possible to distinguish between the various factors causing the tonal patterns—intrinsic soil color, composition of soil, moisture content, vegetation and cultural effects. The advantages of color infrared were better haze penetration and more accurate delineation of drainage conditions and vegetative growth. Also, as previously noted, finer details, identification of smaller objects, and greater accuracies were obtained from color photography than from equivalent black-and-white photography at the same scale. A further advantage of color photography was that special filters could be used in the viewing stereoscope to increase the contrast among features of interest. This made it much easier to delineate and map soil differences.

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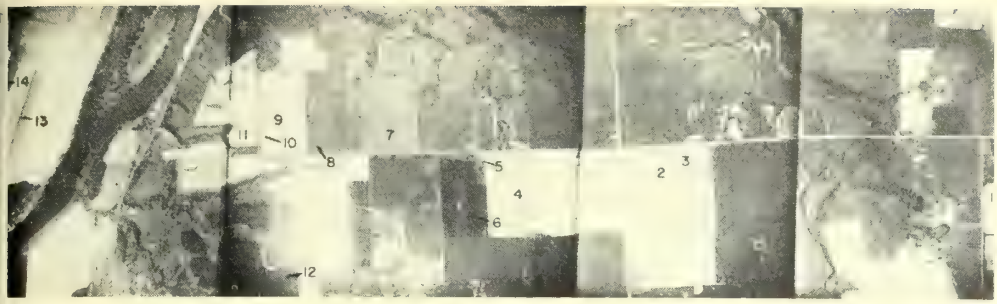
[7] O. W. Mintzer, Summary of *Report on Evaluation of Photo Interpretation Data from Phoenix, Arizona Test Site* (see acknowledgments).

[8] Paper by D. J. Barr and R. D. Miles, presented at Highway Research Board (see acknowledgments).

[9] Report by L. F. Dellwig, presented at Highway Research Board (see acknowledgments).

[10] M. F. Sheridan's comments in NASA technical letter (see acknowledgments).



Point	*Soil or rock unit*	*Condition*
1.	**Thick loess/glacial till**	**High position—bare**
2.	**Glacial till**	**High position—plowing in progress**
3.	**Glacial till**	**High position—recently plowed**
4.	**Glacial till**	**High position—plowed a few days ago**
5.	**Sandstone**	**Small exposure**
6.	**Glacial till**	**Covered with winter wheat**
7.	**Glacial till/sandstone**	**Pasture—sandstone exposed in places**
8.	**Glacial till/sandstone**	**Pasture**
9.	**Glacial till**	**High position**
10.	**Glacial till**	**Depression**
11.	**Glacial till**	**High position**
12.	**Glacial till**	**Covered with winter wheat**
13.	**Flood plain**	**Plowing in progress**
14.	**Flood plain**	**Recently plowed**

Figure 5.—Location of points measured on multichannel imagery.

Mintzer (*5*), Chaves and Schuster (*6*), Anson (*7*), and others have similarly indicated the advantages of color photography. They have shown examples where many items of interest, such as soils and rocks, location of construction material, delineation of landslides and swampy conditions, and others, are more accurately delineated on color photography.

The use of radar and color photography, individually, are of great value in the highway location stage; however, the most promising of all the newer techniques is the multisensor approach in which several different regions of the spectrum are sensed simultaneously. In principle, each object is unique in its response throughout the electromagnetic spectrum. Therefore, it should be possible to delineate the different objects by their varying responses in the different regions. This principle is demonstrated by the examples illustrated in figures 4, 5, and 6.

In figure 4, several sensor forms illustrate the appearance of drainage patterns in different regions of the spectrum. The nighttime infrared image provides the clearest view of the stream and its intricate bends, but tributary creeks are difficult to delineate on this image. The most complete information on the drainage pattern is obtained by analyzing the images of all the sensor forms.

Figure 5 is an aerial photographic mosaic showing the conditions existing at the time the multisensor data were obtained. The location and description of the various terrain features are described in this illustration.

Figure 6 shows the normalized spectral response signatures developed from the multisensor data for the various terrain features noted in figure 5.

The multisensor data for developing the signatures were obtained wlth the multichannel sensor at the University of Michigan. This sensor simultaneously obtained up to 18 channels of data, ranging from the near ultraviolet to the far infrared regions. All the channels were at the same scale, resolution, and format. Thus, by making densitometric measurements of particular terrain features on each channel, and normalizing these to the darkest and lightest tones present, the spectral response signatures shown in figure 6 were developed.

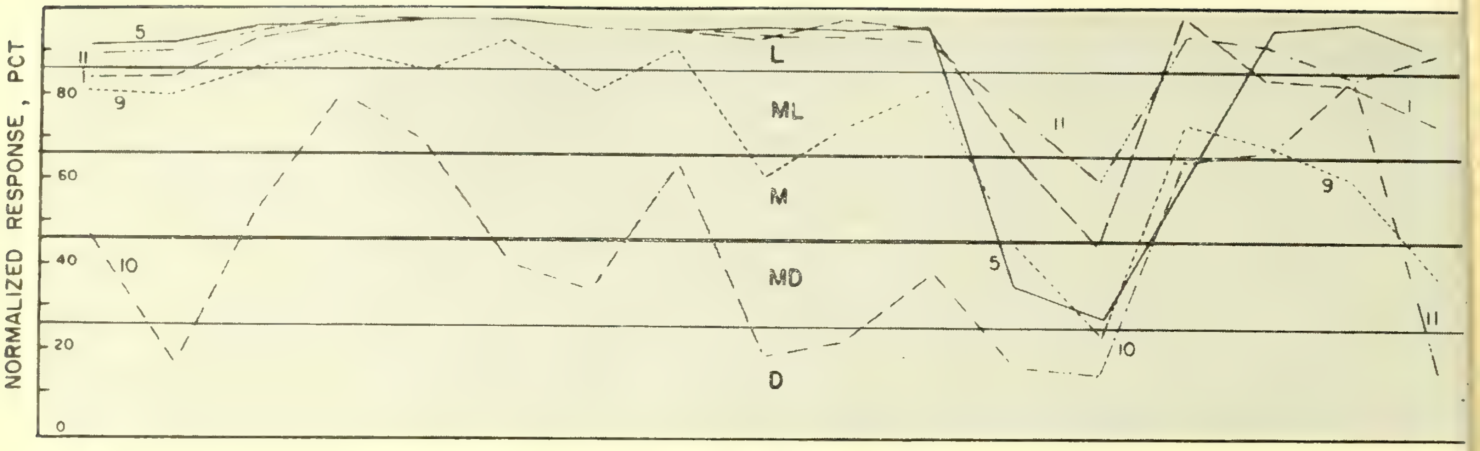
The curves in figure 6 are divided into three groups. Figure 6(a) includes spectral response signatures for bare soils and rock units; figure 6(b) includes spectral response signatures for bare soils, in which tones vary from those in figure 6(a) because of farming practices; and figure 6(c) includes spectral response signatures for various vegetation conditions present in the area. The five bands delineated in figure 6 (L) light, (ML) medium light, (M) medium, (MD) medium dark, and (D) dark are qualitative ratings. They were used to compare the relative tones of the multichannel imagery with those of other forms of photography and imagery for which comparative measurements were not possible.

The spectral response curves in figure 6(a) demonstrate the similarities and differences present for sandstone (curve 5); glacial till soils of various topographic positions, eroded slope (curve 9), depressional area (curve 10), high topographic position (curve 11); and glacial till soil overlain by 4–5 feet of loess (curve 1). All these units can be separated because of distinct differences in various portions of the spectral region. For example, curves 1, 5, and 11, are similar throughout the visible region, but curve 5 shows a darker tone than curves 1 and 11 in the photographic infrared region, whereas, curve 11 shows a darker tone than curves 1 and 5 in the far infrared region. Curves 9 and 10, similarly, have distinct differences to aid in separating them from each other and from the other units.

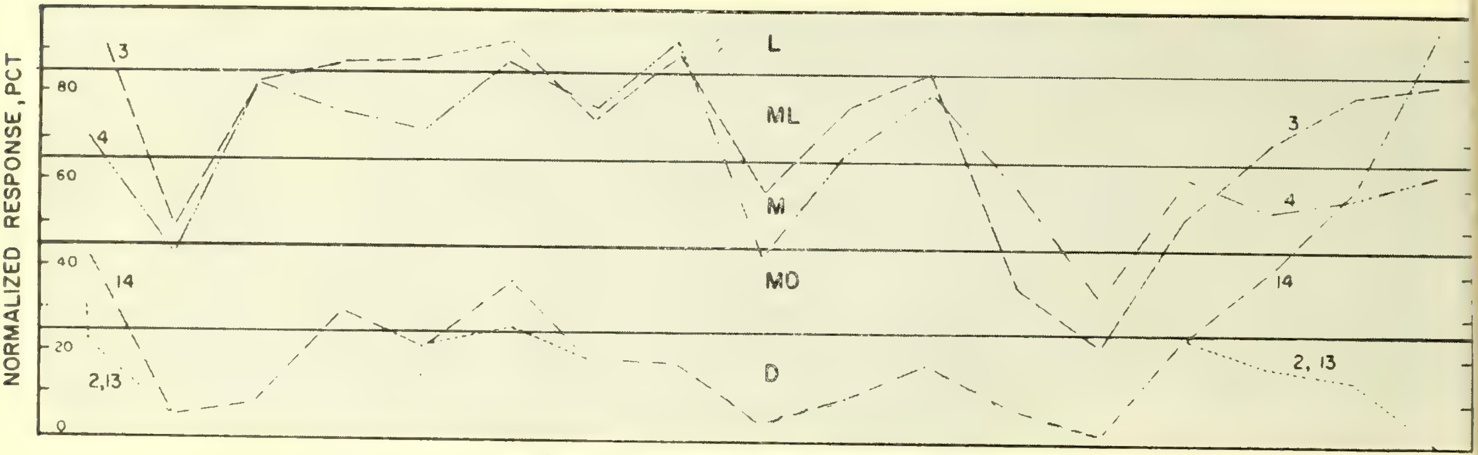
The effect of farm practices on the tones obtained in the different bands is demonstrated in figure 6(b). All these curves represent soils recently plowed. Curve 4 represents a field plowed a few days before the flight. Curves 3 and 14 represent fields plowed the morning of the flight; flight was performed in the afternoon. Curves 2 and 13 represent areas being plowed during the flight, or a very short time prior thereto. Soils represented by curves 2, 3, and 4, are glacial till soils predominantly in the high topographic position, and those by curves 13 and 14 are sandy soils of the flood plains.

The effects of the plowing are to expose, at the surface, the wetter and darker colored subsoils. When the soil is first turned over, the moisture effect is the controlling factor; tones are darker in all bands regardless of texture (curves 2 and 13). As these soils dry out, the effect of moisture is decreased and that of soil color becomes prominent (curves 3, 4 and 14). It can be noted that curves 3 and 4, drying ½ day and 2 days, respectively, are similar and both resemble curve 9, figure 6(a), the eroded glacial till soil in which the subsoils are exposed.

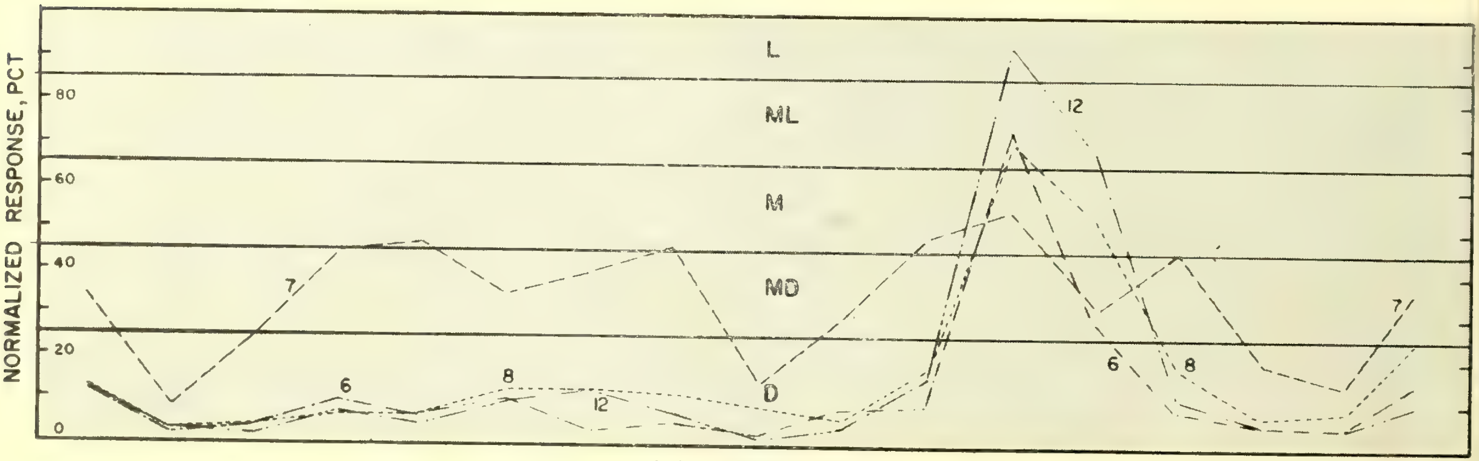
The last group of curves in figure 6(c) shows the differences in spectral responses signatures for various vegetation conditions. Curves 6 and 12 represent fields of winter wheat, and curves 7 and 8 represent pasture fields. It is noted that pasture fields can generally be distinguished from winter wheat by lower reflectance in the photographic IR (infrared) region. It is further noted that all the curves in this figure indicate that the presence of vegetation results in dark or medium dark tones in all bands except the photographic IR. Since the previous curves for soils indicate



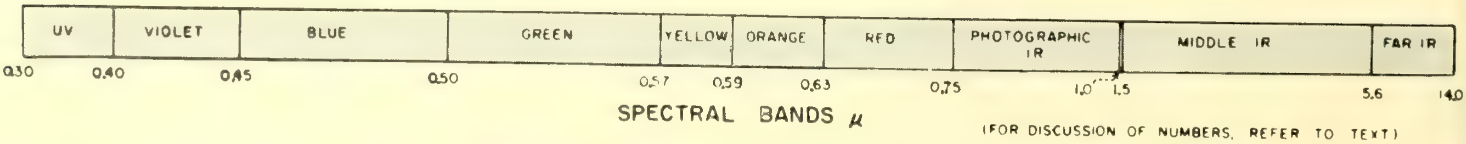
(a) SOILS AND ROCK UNITS



(b) EFFECTS OF FARMING PRACTICES



(c) VEGETATION



(FOR DISCUSSION OF NUMBERS, REFER TO TEXT)

Figure 6.—Spectral response signatures for various terrain features.

that soils have low response in the photographic IR, this is an excellent band for distinguishing tonal effects caused by vegetation from those caused by soils.

The differences in response between the pasture fields shown in curves 7 and 8 is that the field containing point 7 has bedrock close to the surface, and its influence is indicated by the light streaks in the field (see fig. 5). This affects the overall tonal response, resulting in slightly lighter tones. The differences between the fields of winter wheat shown in curves 6 and 12 are a little more difficult to explain. These curves are similar in all bands but the photographic IR. In that band, curve 6 is darker. From investigation of this phenomena in the field, it was determined that field 6 was planted 2 weeks earlier than field 12. In addition, it was discovered that in the previous year this field had been planted in corn and field 12 had been planted in a low cover crop. A botanist suggested that the tonal patterns may reflect vegetation differences because of varying nitrogen levels in the soils. This could not be verified, but similar effects of previous planting history on variations in tonal patterns, obtained for similar crops, has been reported by C. E. Olson.[11]

The example demonstrates the variety of terrain features, important in highway location, that can be interpreted from multisensor coverage, particularly multichannel. The use of the multichannel sensor provides the potential for development of a new technique for automatic interpretation of terrain features, which will be significant to highway engineering. Because the data collected on each channel can be normalized to a common basis, it is possible to compare the information on all channels and arrive at typical tonal signatures for different objects.

The data shown in figure 6 demonstrate that most of these features can be differentiated by using just five broad tonal bands and three to four spectral regions. Thus, most of these features could be differentiated when working with 5^4 or 625 tonal combinations. By again using just five tonal bands and 15 channels, the number of tonal combinations available is more than thirty billion. This should be sufficient to distinguish among most objects of interest. For these comparisons, computers will have to be utilized. This approach is being investigated by the Bureau of Public Roads as well as by other research groups (*8*, *9*).

Construction Surveys

Remote sensing techniques are applicable to the construction stage for obtaining information to assist in the following operations: (1) estimation of contract quantities—clearing, rock excavation, haul distances; (2) determination of construction progress; and (3) ascertainment of damage caused by construction to adjacent property, vegetation, and drainage. As much detailed information is required in the construction stage and because it is necessary to distinguish between numerous items, the use of natural and infrared color photography and multisensor imagery offer the best approach. The color enables one to discriminate and positively identify objects, while the multisensor imagery makes it possible to identify and delineate various items for estimation of quantities.

Maintenance Surveys

Maintenance surveys are performed to evaluate the existing condition of the highway system and facilities, and to determine what corrective measures are needed to bring the system up to the original or an acceptable condition. Included in this stage are surveys determining damage caused by water, wind, landslides, or subsidence.

As in the previous stage, detailed information is required about particular sites, and general information and quantities are needed for large regional areas. For example, information is needed on areas damaged by floods, as well as on details of damage occurring at particular sites. The remote sensors most applicable to this stage include the use of natural and infrared color photography and multisensor imagery. These sensors should provide the detailed information along selected sites, in addition to general and quantitative information for regional areas.

Summary and Costs

In the discussion of the potential applications of remote sensing techniques to the various stages of highway engineering, it is apparent that no single technique, or even combination of techniques, is applicable to all stages. In addition, the information obtained in one stage is not always applicable to other stages. Where large regional areas are to be evaluated, satellites, radar, and other sensors providing broad coverage, are applicable. Where details, quantities or delineation of objects, materials, soils, geology, etc., are needed, a combination of multisensor imagery and color photography appears to be more fruitful. As both regional and detailed information are needed in almost every stage of highway engineering, it is apparent that combinations of these two groups of coverage will be required to completely evaluate all the pertinent factors. In addition, to meet the requirements for each stage, periodic coverage is important. These requirements raise the important question of cost.

The cost of multisensor flights has been estimated at $1,000, or more, an hour. However, there is little information in the literature showing actual cost figures. Information is also lacking on comparisons of costs for performing surveys by remote sensing techniques and by conventional methods. Although remote sensing techniques might provide more information and more accurate information, it would not be applied if the cost becomes prohibitive or is much greater than conventional methods. To determine this cost factor, more research is required.

Public Roads Remote Sensing Research

The Bureau of Public Roads has developed a research program to promote investigation of the use of aerial remote sensing techniques to highway engineering, in general, and to identify natural materials, in particular. The program is entitled *Optimizing Utilization of Natural Materials by Means of Remote Sensing Techniques* and is a part of the *National Program for Research and Development in Highway Transportation.* Investigations in this research program are to be conducted in three principal areas: identification of natural materials by aerial remote reconnaissance techniques; identification and evaluation of natural materials by field remote reconnaissance techniques; and automatic identification of natural materials.

The goal of this program is to establish test sites in several areas of the United States representing different environmental conditions. Attempts will be made to develop diagnostic characteristics and patterns for various natural materials, both locally and on a regional basis. The research in these three areas is to be conducted by a combination of staff research, contract research, and cooperative research between State highway departments and the Bureau of Public Roads' Highway Planning and Research Program (HP&R). In the overall research program, Public Roads also hopes to cooperate with other Federal and private research organizations interested in remote sensing, so as to further the knowledge in this field and avoid costly duplication whenever possible.

Several HP&R studies, in which aerial remote sensing techniques are being evaluated, are in progress. A study entitled *Annotated Aerial Photographs as Master Soils Plans* is underway at Purdue University. Color photography and multisensor imagery are being evaluated in this study for soils mapping. Phase A of the study indicated that the optimum combination of sensors for detailed soils mapping in Indiana was natural color photography and multichannel imagery (*2*). In arriving at this conclusion, five different types of aerial film (three scales), two different infrared sensors, two different radar sensors, a multichannel sensor, and a multiband camera were evaluated over a 13-month period. In Phase B of this study, the optimum combination is being evaluated further along a 70-mile route location in Indiana. As part of this second phase, cost comparisons of the remote sensing technique versus standard field methods for determining soil and soil conditions pertinent to highway location and design are being evaluated.

Cooperative HP&R studies, evaluating the use of color photography for various highway applications, are in progress in Maine, Florida, and Ohio. A study is also in progress in California, where infrared imagery is being evaluated to determine information on types and identification of soils as a suitable source for construction material.

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[11] Paper by C. E. Olson, Jr., presented at the American Association for the Advancement of Science meeting (see acknowledgments).

The Public Roads staff research in remote sensing is concentrating on the problem of automatic identification. Based on multichannel data and color photography, investigations are underway to develop techniques for identifying various soils and soil conditions, and for automatic identification of various road surfaces. Work is also in progress on increasing the accuracy of arriving at standard descriptions of colors present in color photography by means of densitometric measurements. This method provides a means of automatically differentiating items which can be separated by color differences. This technique has been described by Gourley, Rib, and Miles (*10*) and Rib (*11*).

A research contract with industry to determine the feasibility of the development of an area traffic surveillance and control system is also in progress. The scope of this contract includes investigations into the feasibility of evaluating significant traffic parameters to specific tolerances—for example, speed in miles per hour ±0.5 m.p.h. per vehicle—from ground base systems, airborne systems and orbiting satellites.

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Kaolin-Lime-Water Systems
Part 1—Strength and Rheological Properties

*BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS*

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The strength characteristics and flow properties of well-defined kaolin-water systems were evaluated in the study reported in this article. The systems contained different percentages of hydrated lime and were cured at room temperature for periods up to 15 days. Unconfined compressive strengths were measured, and flow properties were evalutated using a cone-plate viscometer operated over a range of shear rates.

Unconfined compressive strength was generally improved by lime additions; significant improvement was noted for even very small additions, such as one-half to 1 percent. For the two curing periods used, 3 days and 15 days, most of the strength had been developed by the end of the 3-day period.

The viscosity, or consistency, of the kaolin-water systems was markedly increased by the addition of lime; additions as small as 0.8 percent effected considerable change. The effect was immediate and, as indicated by the rheological data, was complete in less than 10 minutes.

Introduction

BENEFITS DERIVED from treating soils, clays, and other materials with lime have been recognized for many years. The use of lime for soil stabilization, that is, to increase strength, durability, and resistance to deformation of native soil, has been the subject of many laboratory and field investigations, the results of which have been summarized in numerous reports (*1–6*)[2], by Eades,[3] and by Mateos (*7*). These reports include results of investigations on the physical and chemical characteristics of lime-stabilized soils and clays.

Mateos (*7*) emphasized the importance of the lime retention concept proposed by Hilt and Davidson (*4*), wherein it is hypothesized that a certain initial concentration of lime, "lime retention value," must be exceeded before pozzolanic action can occur. In this concept, lime concentrations at and below this initial concentration ameliorate the soil by changing properties, such as plasticity and shrinkage, without improving strength; strength development occurs after this initial lime requirement is satisfied.

In contrast to the lime retention concept, Baldino [4] and Kiek (*8*) demonstrated that pronounced improvements in strength characteristics of some soils and clays can be effected by additions below their probable lime retention values.

The study reported here was initiated to investigate further the concentration of lime required for strength development. Well-characterized kaolin-lime-water systems were evaluated with respect to the effects of lime additions on strength and rheological properties. The percentages of lime used extended over a considerable range of concentration, including amounts less than the potential lime retention values of the clays under study. Assuming that the lime retention concept is valid, little change in strength properties or flow behavior would be expected from additions of lime that are less than the critical amount.

Materials

Three kaolins previously (*9*) characterized as to mineral composition, cation-exchange capacity, surface area, and crystallinity, were selected for study. Sample designations B, H, and E are the same as those used by previous investigators (*10–12*). Properties of kaolins are listed in table 1.

Moisture-Density Relations

In order to approximate optimum conditions for the fabrication of specimens for strength tests, it was necessary to determine moisture-density relations for various clay-lime-water mixtures. As only a small amount of each of the three clays was available, the compaction tests were miniaturized and performed using kaolin B only. Because of the close similarity of the three kaolins, as shown by the data in table 1, it was assumed that the compaction

Table 1.—Properties of clays studied

Property		Kaolin		
		H	B	E
Surface area	meters²/gram	12.10	11.70	11.80
Crystallinity index [1]	021/060	0.24	0.26	0.19
Cation-exchange capacity	m.e./100g.	2.14	2.24	2.30
Indicated mineral composition:				
X-rays		Kaolinite	Kaolinite	Kaolinite
Electron micrographs		Kaolinite	Kaolinite	Kaolinite
pH $\left(\frac{\text{Clay}}{\text{Water}} \frac{1}{20}\right)$		5.3	5.2	5.2
Liquid limit		45	46	48
Plasticity index		14	15	18

[1] The crystallinity index is determined from the ratio of the intensities of 2 X-ray diffraction peaks. The $(02\bar{1})$ peak is affected by changes in crystallinity whereas the (060) peak is not.

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[1] Presented at the 70th annual meeting of ASTM, Boston, Mass., June 1967.

[2] The italic numbers in parentheses identify the references listed on page 43.

[3] J. L. Eades, *Reaction of $Ca(OH)_2$ with Clay Minerals in Soil Stabilization*. Ph. D. Thesis, University of Illinois, 1962.

[4] J. D. Baldino, *Analysis of a Factorially Designed Experiment on Lime Stabilized Pure Clay Minerals*. C. E. 440 Report, Fritz Engineering Laboratory, Lehigh University, 1963.

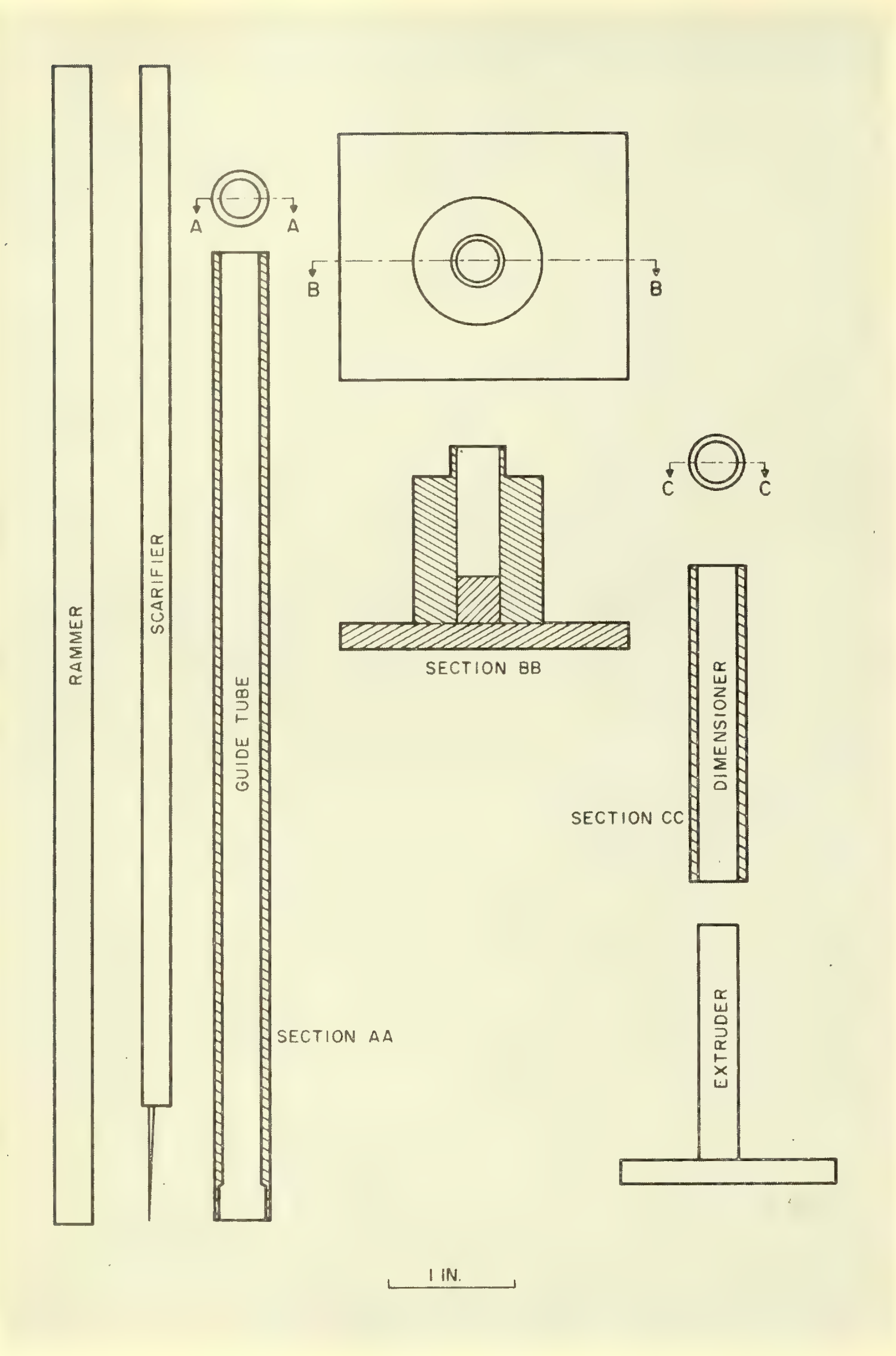


Figure 1.—Molding apparatus.

results for kaolin B would also be representative for kaolins H and E. Drawings of the molding apparatus and accessories are shown in figure 1. All parts of this equipment were made of brass except the mold which was made of teflon.

Clay for the compaction experiments was powdered to pass a No. 80 sieve and dried. For each composition, the required clay and $Ca(OH)_2$ were weighed, mixed on glazed paper, and placed in a small plastic bag. Distilled water was added, and the bags were sealed and thoroughly kneaded by hand. The mixture was immediately removed and compacted using three layers, which were each tamped eight times. This technique was satisfactory for determining optimum moisture content, and produced densities somewhat higher than Proctor densities obtained by Eades[3] for similar materials.

After fabrication, specimens were weighed and dried at 110° C., and densities were calculated from specimen weights and dimensions. Plots of density versus moisture content for different lime concentrations are shown in figure 2. As expected, the curves indicate a decrease in density with increasing lime concentration. A slight increase in optimum moisture content also occurred as the lime content was increased. On the basis of the results shown in figure 2, subsequent compositions for strength specimens were prepared to yield a measured moisture content of 22 percent, dry weight basis.

Strength Experiments

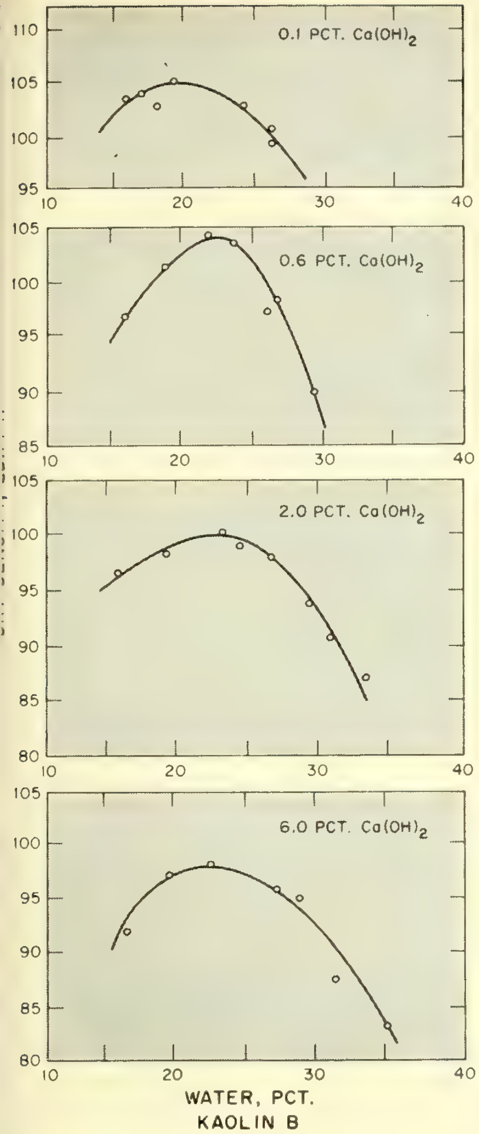
Strength experiments were performed on specimens prepared using kaolins H and E. Four replicate specimens of the following compositions were prepared: 0.0, 0.1, 0.2, 0.6, 0.8, 2.0, and 6.0 percent lime on the dry-weight basis. The required amounts of clay, lime, and water were placed in plastic bags and were kneaded and compacted using the same molding equipment and compaction procedures that were employed for the moisture-density tests. Specimens were cured in a high humidity chamber at room temperature (23°–26° C.) for periods of 3 days and 15 days.

The compressive strength was determined using the apparatus shown in figures 3 and 4. Load was applied at a constant strain rate of 0.005 cm./min, and the chart speed was 0.5 cm./min. Loading sensitivity was ±10 grams. The unconfined compressive strength was computed from the rupture load and the cross-sectional area of the specimen.

Viscosity Experiments

A cone-plate viscometer equipped with a flow curve recorder was used in the viscosity experiments. The cone and plate arrangement and sample positioning are shown schematically in figure 5. A photograph of the viscometer with accessory apparatus is shown in figure 6. A truncated cone, 4 centimeters in diameter, was used in all of the experiments. The cone was driven by a variable speed motor through an electrical torque dynamometer that measured the viscous traction caused by the sample. The amount of traction was indicated on an X-Y recorder where shear rate (proportional to the driving speed) was plotted against torque (proportional to the shear stress). Principles of operation of the viscometer are fully described in the literature (*13, 14*).

Required amounts of kaolins H and E were screened to pass a No. 80 sieve and dried at 110° C. Small portions of these clays were then weighed on glazed paper, and predetermined amounts of lime—concentrations of 0.0, 0.1, 0.2, 0.6, 0.8, 2.0, and 6.0 percent on the dry-weight basis—were added and mixed. The mixtures were placed in small plastic bags and distilled water was added to give a concentration of 100 percent water, dry-weight basis. The bags were then sealed and stored under the same humidity and temperature conditions as the strength specimens. For each



igure 2.—Moisture-density relations for kaolin B.

lay, two specimens were prepared at each ime concentration to provide separate specinens for each of the two curing periods of 3 lays and 15 days. After the prescribed curing, he sample bags were removed from the hunidity chamber and thoroughly kneaded by and. Each clay-lime-water mixture was then livided into two parts, one for a verification of the moisture content, and the other for low property evaluation. The flow behavior vas determined using a predetermined schedile of shear rates and times. For most mixtures n equilibrium curve was obtained after two or three cycles, each cycle consisting of an utomatically-programed acceleration from ero to the maximum shear rate in a given ime followed by a deceleration to zero in the ame length of time. During the operation of he viscometer the specimen mixtures were naintained at a temperature of 25±0.2 legrees C.

Table 2.—Strength of clay-lime specimens

Concentration of $Ca(OH)_2$ added to clay specimens	Strength of clay specimens [1]				Average strength for all specimens at each lime concentration
	Kaolin H		Kaolin E		
	Cured 3 days	Cured 15 days	Cured 3 days	Cured 15 days	
Percent	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*	*p.s.i.*
0	186	208	193	160	187
0.1	215	180	187	164	186
0.2	208	182	198	178	192
0.6	207	186	205	166	191
0.8	214	190	209	202	204
2.0	206	238	196	214	214
6.0	220	243	211	213	222

[1] Each strength value is the average of four replicates.

A separate series of specimens was prepared o study the effect of short curing periods. These specimens, containing 2 percent lime and 100 percent water (dry-weight basis), were cured for periods up to 32 hours.

Results of Strength Tests

Results of the strength tests are given in table 2. An analysis of variance of the results in columns 2 to 5 was performed and is given in table 3. The interaction mean squares given in table 3 are of the order of magnitude to be expected from the variance computed using ranges of the individual determinations. Individual strength results, although not listed, were used to compute a standard error for

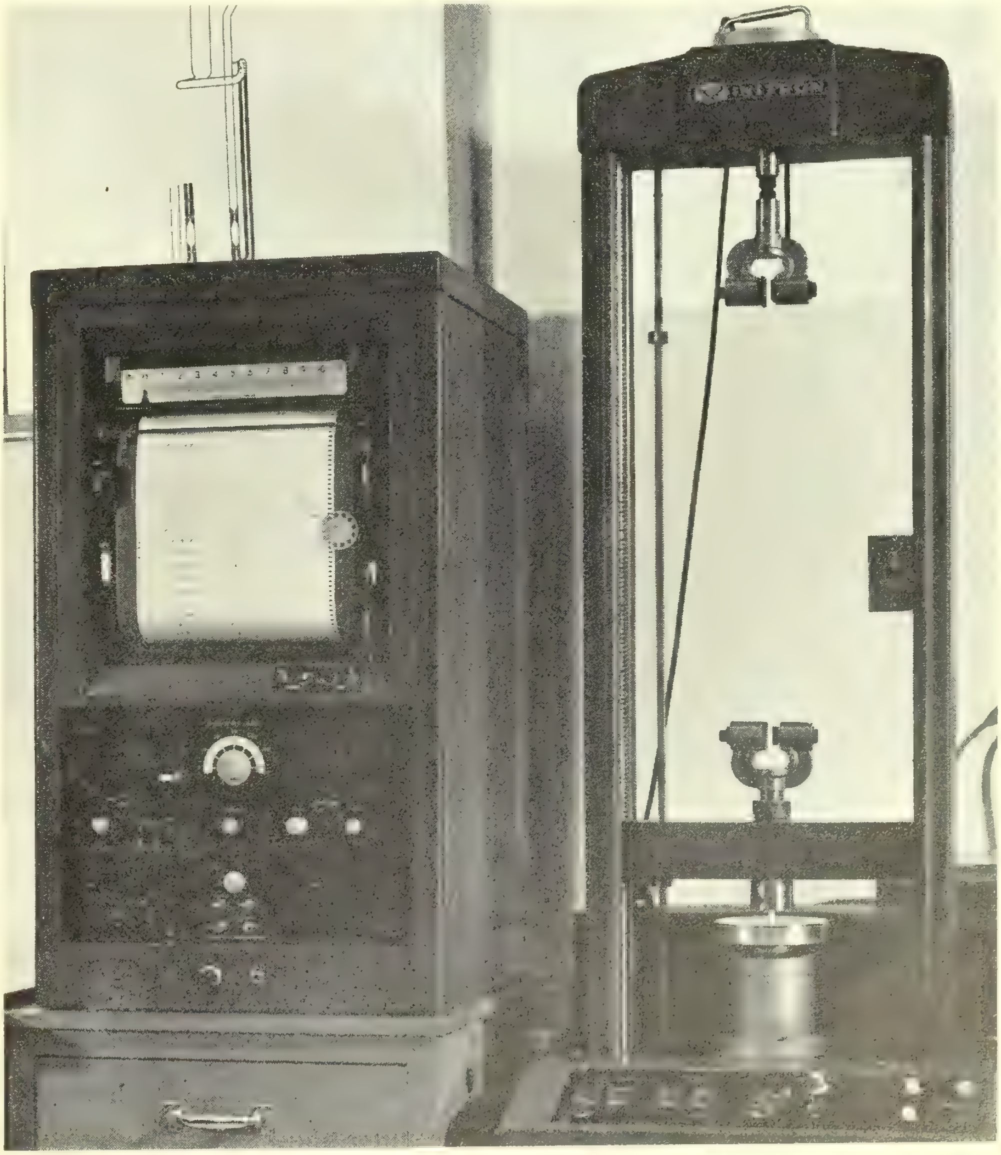


Figure 3.—Apparatus used in determining compressive strength.

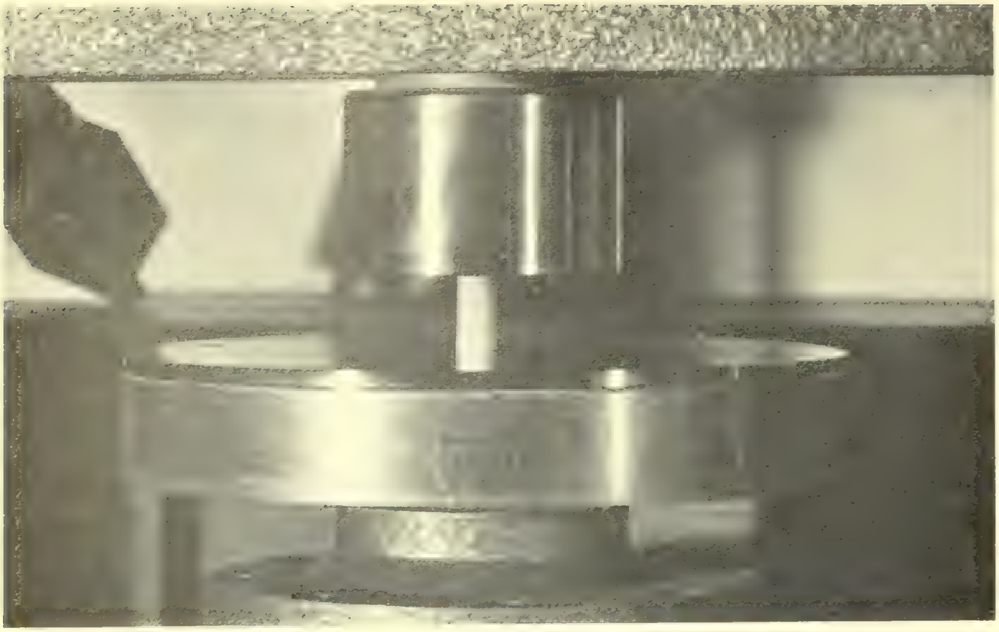


Figure 4.—Detail of loading arrangement showing sample positioning.

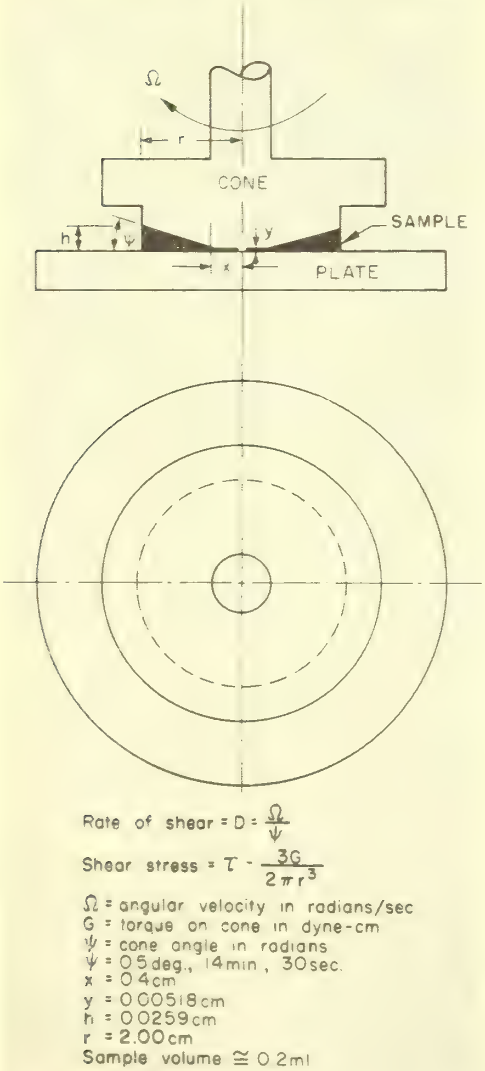

Ω
r
CONE
SAMPLE
h
ψ
y
x
PLATE
Rate of shear = D = Ω/ψ
Shear stress = τ - 3G/2πr³
Ω = angular velocity in radians/sec
G = torque on cone in dyne-cm
ψ = cone angle in radians
ψ = 0 5 deg., 14 min , 30 sec.
x = 0 4 cm
y = 0 00518 cm
h = 0 0259 cm
r = 2.00 cm
Sample volume ≅ 0 2 ml

Figure 5.—Cone-plate viscometer details (not to scale).

means, and this standard error amounted to 13. This gives a variance of 169 for averages, which is in reasonable agreement with independent estimates of variance computed for various interactions in the analsysis of variance (table 3). The analysis indicates significant differences between clays and between lime concentrations and, importantly, shows that there are no significant interactions between clays and lime, between time and lime, or between clays and time. Therefore, because no interactions were detectable, one is justified in computing a concentration average in which the strengths at each lime concentration for the clays at both curing times are average. This average, based on 16 determinations, is given in column 6 of table 2.

To evaluate further the relationship between concentration of lime and strength, least-squares polynomials were fitted to the data (columns 1 and 6 of table 2). Results indicate a good fit for a second degree equation—$Y=186+18X-2X^2$, index of determination $(r^2)=0.95$, standard error of estimate for $Y=3.86$ p.s.i. The curve calculated from this equation is plotted along with the experimentally determined points (open circles) in figure

Higher degree equations were also fitte but the fitting did not result in any improvement. Table 4 gives an analysis of varianc for regression of the various equations fitte

The results of the least-squares fitting an the analysis of variance indicate the followin

- An appreciable improvement in strengt upon the addition of lime.
- Strength improvement begins at low concentrations of lime.
- Initial gain in strength is rapid but begin to taper off at about 2 percent lime.
- A second degree equation can adequatel account for the variation of strength as th lime concentration is changed.

Viscosity Results

Flow curves for the different systems studie are given in figures 8–11. In general, thes curves indicate an increase in the rigidity o the systems as the lime concentration i increased. This effect is evident from a comparison of the dynamic yield points which ma be obtained by visually extrapolating the mor or less straight-line portions of the curve back to zero shear rate. Increase of the yiel points of the systems, as the percentage o lime is increased, indicates that lime produce a strengthening of the clay-water structure Because of the difficulties in accurately drawing tangents to the curves, tangents were not used in calculating apparent viscosities instead, the flow curves were analyzed using the power law equation described later.

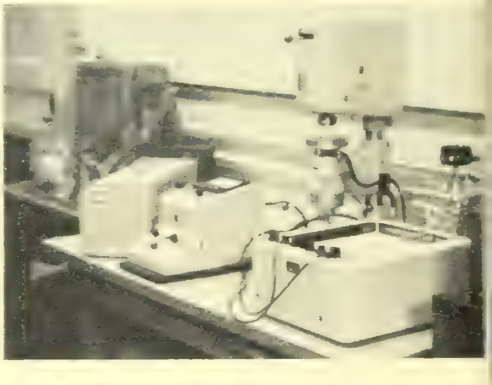


Figure 6.—Cone-plate viscometer with flow curve recorder used in viscosity experiments.

Table 3.—Analysis of variance—strength results

Source of error [1]	Sum of squares	Degrees of freedom	Mean square	*Test* degrees of freedom [2]	F ratio [3]
Clays	1249	1	1249	1 19	5.556*
Times	671	1	671	1 19	2.985 n.s.
C X T	153	1	153	1 6	0.955 n.s.
				1 18	0.688 n.s.
Lime	4707	6	784.5	6 18	3.429*
L X C	2072	6	345.3	6 6	2.155 n.s.
L X T	1085	6	180.8	6 6	1.128 n.s.
L X T X C	961	6	160.2		
Total	10898	27			
POOLED ERROR (1)	4118	18	228.77		
POOLED ERROR (2)	4271	19	224.79		

[1] C=Clays=2; L=Lime Concentration=7; T=Times=2; LTC=28.
[2] Test degrees of freedom indicates which error term was used in testing.
[3] *—Significant at 5% level; n.s.—not significant.

The curves for kaolin H (figs. 8 and 9) are from determinations made on an 80-second schedule; that is, 40 seconds to uniformly accelerate the spindle driving the cone to the maximum r.p.m. of 1000, and 40 seconds to decelerate to zero. The curves obtained during acceleration usually coincided with those obtained during deceleration. This behavior indicates that the structure rapidly breaks down when the shear rate is increasing and, during the deceleration portion of the cycle, reforms at the same rate at which it was broken down, returning to the original structure at zero shear rate. Recycling of the specimens produced curves that were almost identical to the original.

The curves for kaolin E, shown in figures 10 and 11, were run on a 20-second schedule. With this clay, some hysteresis—slight opening up of the up-and-down curves—was observed; this hysteresis is not evident in the figures, because only the downcurves are illustrated.

Figure 12 gives long–log plots of viscosity data obtained for the various mixtures of kaolin H that had been cured 15 days. These plots, which are typical for all of the systems studied, suggest a power law relation,

$$\tau = kD^N \text{ or } \eta_a = kD^{N-1}$$

Where,

η_a = apparent viscosity (poise)
D = rate of shear (sec^{-1})
τ = shear stress ($dynes/cm^2$)
k = power law constant
N = power law exponent.

The constant k is proportional to the consistency of the system, and N gives an indication of the departure from Newtonian behavior. For $N = 1$, $N > 1$ and $N < 1$, one obtains Newtonian, dilatant, and pseudoplastic behavior, respectively. Results obtained for the power law parameters, N and k, for kaolins H and E, are given in table 5.

The relations between the viscosity parameters k and percent lime are shown in figure 13 for the two clays. This figure indicates that:

- Viscosity is increased at relatively low concentrations of lime.
- For kaolin H, the increase of viscosity progresses fairly regularly up to 6 percent lime; for kaolin E, the viscosity increases fairly regularly up to 2 percent lime, then remains essentially constant upon further additions.
- The degree of crystallinity of the clay may influence the viscosity to a considerable degree, as the more poorly crystallized clay E has a much higher viscosity than kaolin H.

Results of the experiments designed to determine the effects of short curing periods are given in figure 14. The power-law parameters for the curves of figure 14 are given in table 6. It is noteworthy that the consistency parameter k for kaolin E with 2 percent lime is 1755, which is more than double that of the 3-day specimen of the same kaolin without lime (see table 5). It is thus clear that, as far

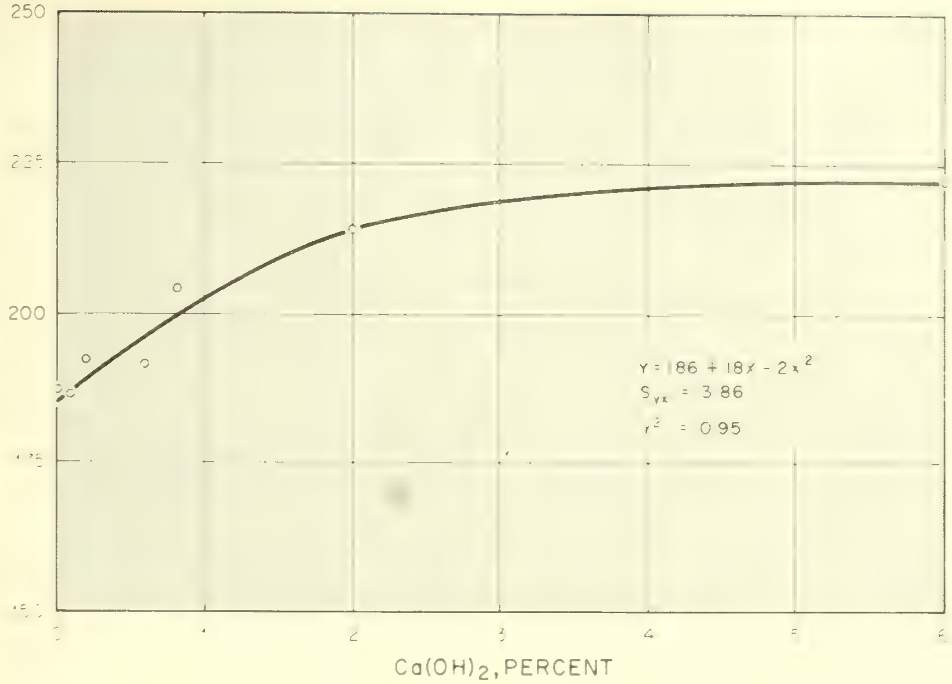


Figure 7.—Relations between unconfined compressive strength and percent lime for kaolins H and E.

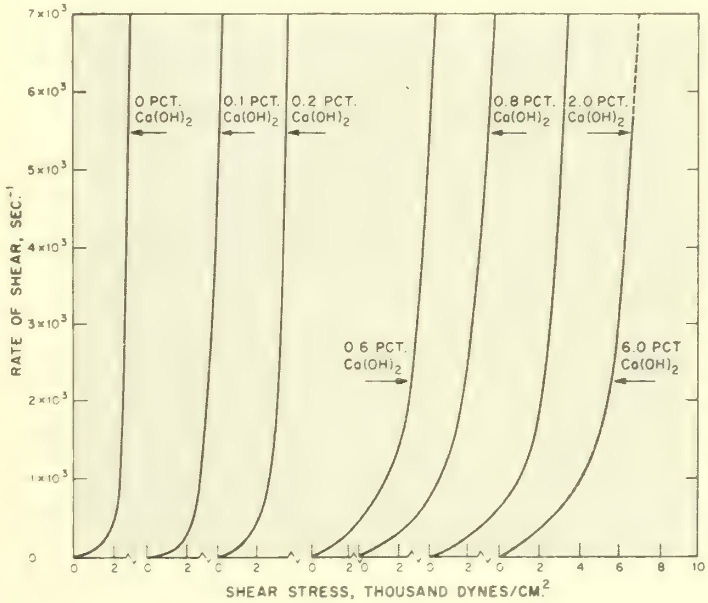


Figure 8.—Flow curves for kaolin H cured 3 days.

Table 4.—Analysis of variance for regression

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Linear	931.22820	1	931.22820	[illegible]
2nd degree	212.97[illegible]	1	[illegible]	14.76*
3rd degree [1]	0.06019	1	[illegible]	[illegible]
4th degree [1]	10.99475	1	10.99475	0.4375 n.s.
5th degree [1]	30.77905	1	30.77905	1.7406[illegible]
Residual	17.68261	1	17.68261	
Total	[illegible]			

[1] Variances for 3rd, 4th, and 5th degree [illegible] against residual or the appropriate pooled variances.

** Significant at 1 percent [illegible]

as viscosity increase is concerned, the effect of lime is almost complete in less than 10 minutes.

Table 5.—Viscosity parameters

Clay	Concentration of $Ca(OH)_2$	Power law parameters			
		Specimen cured 3 days		Specimen cured 15 days	
		Behavioral index, *N*	Viscosity index, *k*	Behavioral index, *N*	Viscosity index, *k*
	Percent				
Kaolin H	0.0	0.24	298	0.26	316
	0.1	0.26	377	0.32	197
	0.2	0.25	412	0.28	303
	0.6	0.26	614	0.28	303
	0.8	0.26	715	0.24	474
	2.0	0.27	644	0.28	632
	6.0	0.22	1016	0.24	954
Kaolin E	0.0	0.24	692	0.25	907
	0.1	0.20	695	0.22	646
	0.2	0.18	919	0.22	646
	0.6	0.24	1376	0.21	815
	0.8	0.21	2002	0.18	926
	2.0	0.18	2442	0.20	2302
	6.0	0.19	2440	0.20	2302

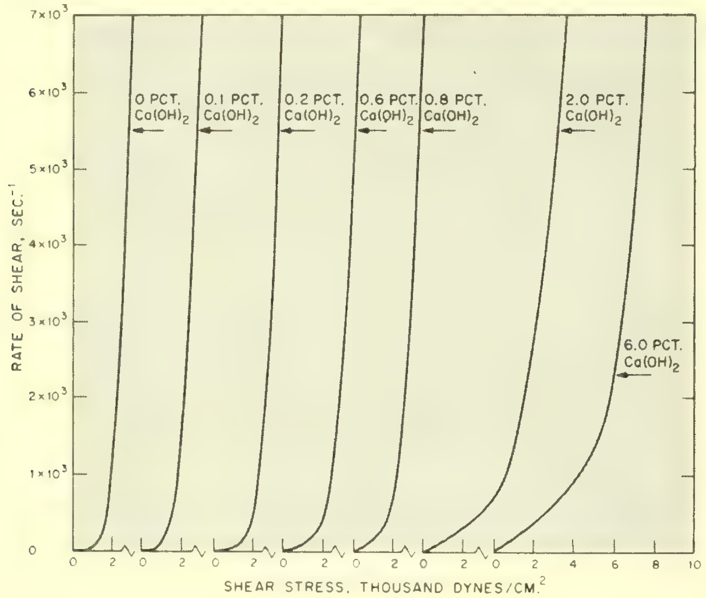


Figure 9.—Flow curves for kaolin H cured 15 days.

Discussion

Previous work (*4*) suggests that the lime retention value of kaolinite clays is usually about four or five times the cation-exchange capacity. In kaolins H and E this would correspond to a lime retention value of nearly 0.4 percent. From the results described (fig. 7), it was determined that strength was developed at lime percentages of the order of 0.1–0.2 percent, which are considerably less than the probable lime retention values of the clays.

As the clays were not calcium saturated, prior to the addition of lime, some of the lime added may have been used up in ion exchange before some additional increment became effective in promoting the development of strength. In other words, if the clays had been calcium saturated, an unequivocal strength response might have been noticed even with lime contents as low as 0.1 percent.

Although cementitious phases were not unequivocally identified in systems containing very low percentages of lime, it seems likely that such phases were present, as considerable interaction of clay and lime is indicated in electron micrographs of replicas of specimens containing only 0.8 percent lime (see Part 2).

The relation between the viscosity parameter and the concentration of lime (fig. 13) indicates that lime has had a pronounced stiffening action. Viscosity increased when even very small amounts of lime were added. At low concentrations of lime, the effect may be one of flocculation and aggradation—similar to that produced in the consistency of ceramic slips when they are cast in plaster molds that have been treated with calcium salts (*15*). Figure 13 also shows that flow properties may be affected by the degree of crystallinity of the clay.

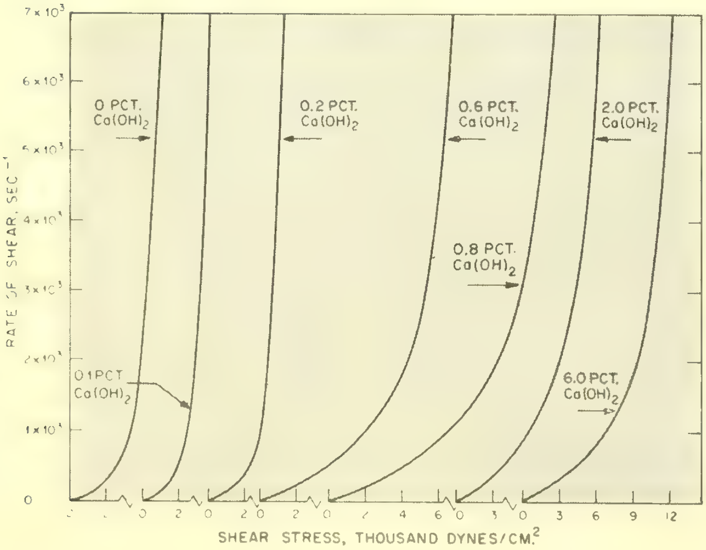


Figure 10.—Flow curves for kaolin E cured 3 days.

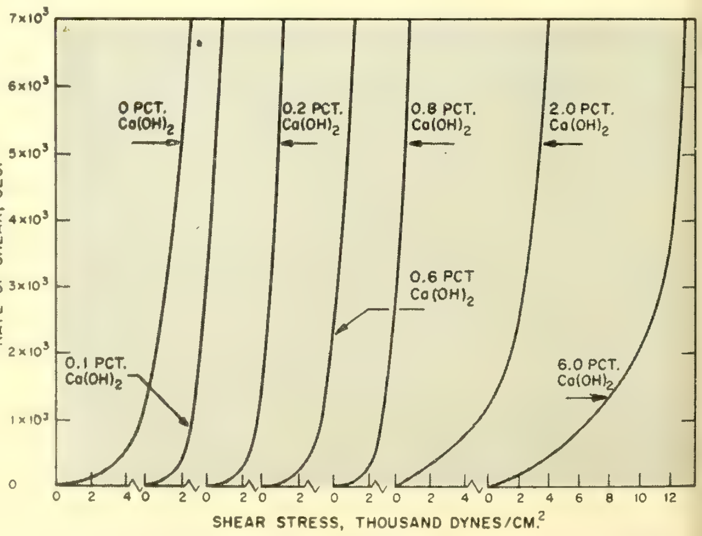


Figure 11.—Flow curves for kaolin E cured 15 days.

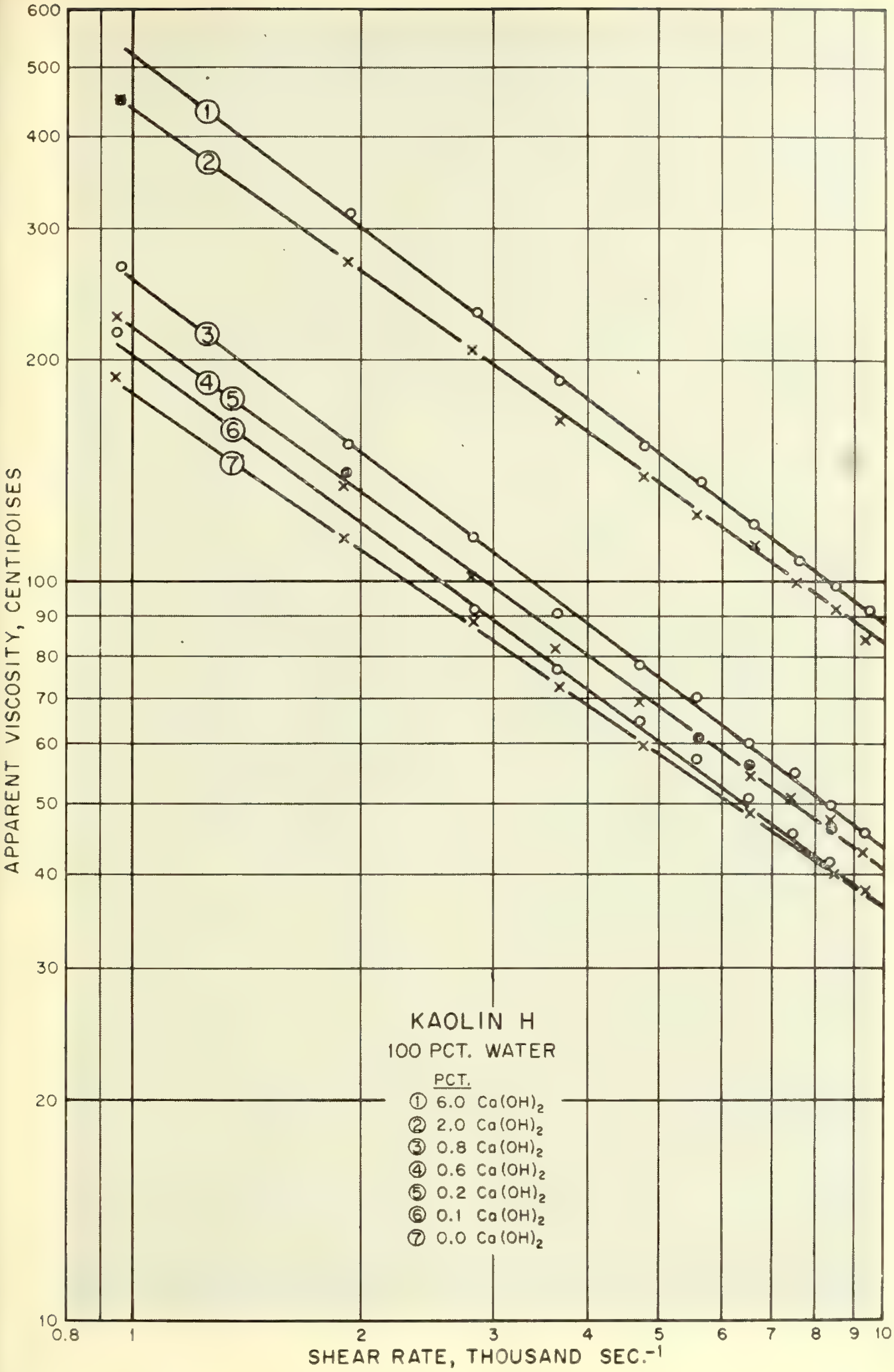


Figure 12.—Apparent viscosity-shear rate plots for kaolin H cured 15 days.

Summary

In the evaluation of strength characteristics and flow properties of kaolin-water systems containing various percentages of hydrated lime and cured at room temperature for periods up to 15 days, unconfined compressive strength of specimens was found to increase as the percentage of lime was increased.

Additions of modest amounts of lime—0.1 to 0.8 percent—produced an immediate increase in viscosity, or consistency; this effect was enhanced as the percentage of lime was increased. After the immediate viscosity increase, there was little further change during periods up to 15 days. For a given clay-lime-water composition the viscosity was higher for the more poorly crystallized clay.

The existence of a critical lime retention value, below which strength and viscosity effects would be minimal, was not revealed.

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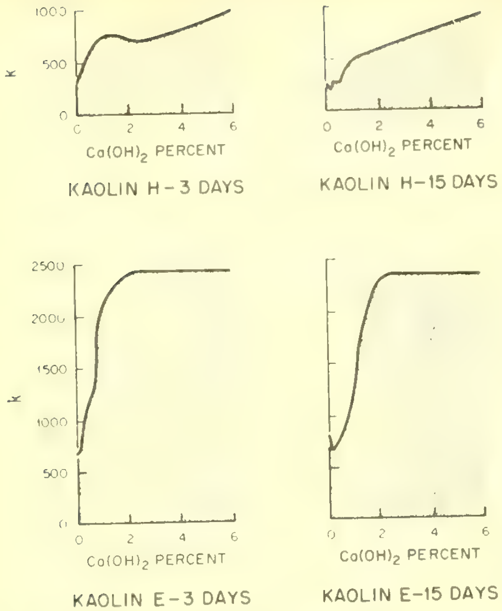

Figure 13.—Relations between viscosity parameter, k, and concentration of lime for kaolins H and E at different curing times.

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Table 6.—Effect of curing time on the viscosity parameters of kaolin E +2% lime

Curing time	Behavioral index, *N*	Viscosity index, *k*
Uncured	0. 21	1755
10 minutes	0. 22	1703
30 minutes	0. 22	1765
2 hours	0. 22	1927
8 hours	0. 20	1745
32 hours	0. 18	1960

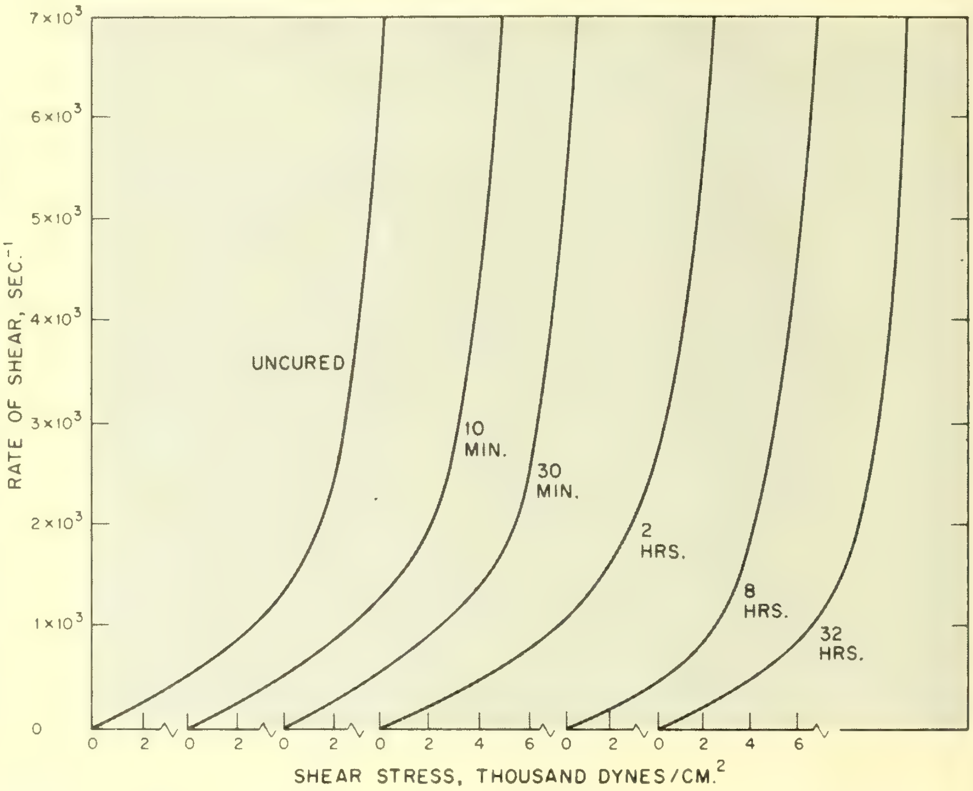

Figure 14.—Effect of curing time on flow behavior of kaolin E, 2 percent lime added.

Kaolin-Lime-Water Systems
Part 2—Electron Microscope Observations

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by [1] W. C. ORMSBY, Research
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Bureau of Standards

Electron optical investigations were made of kaolin-lime-water mixtures that had been cured for various periods. These mixtures, in the form of cylindrical compacts, were recovered from strength experiments. Considerable attack of kaolinite particles by hydrated lime was shown both in electron micrographs of replicas made from fracture surfaces of the compacts and in transmission electron micrographs of powdered compacts; attack occurred both on particle edges and basal surfaces. The formation of new phases—calcium silicate hydrates—was indicated by electron microscopy and confirmed by electron diffraction. The presence of these hydrates is undoubtedly responsible for the stabilizing effects (strength improvements) of lime reported in Part 1.

Introduction

EXAMINATION OF reaction products in clay-lime-water, soil-lime-water, and analogous systems is severely handicapped by the fact that, generally, the products are poorly crystalline or amorphous, and occur in very limited quantities. These problems were recognized by Eades and Grim (*1*)[2], and by Diamond et al., (*2*). In attempts to develop relatively large quantities of reaction product of at least moderate crystallinity, these researchers resorted to very high lime concentrations and, in some instances, elevated curing temperatures. Under these conditions, certain interesting reaction products were detectable by X-ray diffraction, and speculation about reactions at room temperatures and low lime concentrations could be made. However, for intensive study of systems that more nearly approximate those encountered in practice, it is necessary to use a method of analysis other than X-ray diffraction. Electron microscopy and electron diffraction have been used successfully by El Jack and Mitchell[3] in studying soil-cement-water systems, and by Sloane (*3*) in studying clay-lime-water systems. Systems studied by Sloane contained a large excess of water and did not closely approximate conditions that might be encountered in the practical application of lime as a soil stabilizing agent.

The study reported in Part 1 entailed investigation of the strength and rheological properties of kaolin-lime-water systems. The purpose of the investigation reported here was to examine, by electron microscopy and electron diffraction, the microtexture and composition of reaction products in the systems, using mixtures that were recovered from experiments on physical properties.

Materials

Specimens recovered from strength experiments with two kaolins, H and E; were used in the studies; these kaolins are fully described in Part 1. The specimens, originally in the form of cylindrical compacts, 0.35 inches in diameter by 0.70 inches high, had been broken in unconfined compression, immediately dried at 110° C., and stored in airtight containers. For replica studies, a suitable unbroken portion of each specimen was newly fractured, and replicas were made of the fracture surfaces. For transmission electron microscopy and electron diffraction, the specimens were powdered to pass a No. 80 sieve and used as described herein.

Electron Microscopy

Replicas were made using a procedure similar to that outlined by Comer and Turley (*4*), which included the following steps:

(1) Strength specimens were fractured perpendicular to the altitude of the cylinder.

(2) Palladium was vacuum-evaporated onto the fractured surface at an angle of about 30° to the sample surface.

(3) A second palladium evaporation was made at an angle of approximately 30° to the sample surface from the opposite direction.

(4) Carbon was similarly deposited normal to the sample surface.

(5) Specimen was placed in distilled water with replica side up to allow the bulk of the clay sample to slake away.

(6) Replica was then placed in concentrated hydrofluoric acid to remove the last traces of clay; two treatments in hydrofluoric acid were usually necessary.

(7) Replica was washed with distilled water.

(8) Replica was picked up on an electron microscope grid and stored in a desiccator.

Replicas of true fracture surfaces were prepared from mixtures of kaolin E containing 0, 0.8, and 6 percent lime. Replicas were also prepared of pseudo-fracture surfaces and the lateral surfaces of some of the specimens. Pseudo-fracture surfaces are those formed when strength specimens are parted at a layer boundary. These layer boundaries resulted from the manner of specimen fabrication—that is, the specimens were molded using three layers of clay.

Specimens for transmission electron microscopy and electron diffraction were prepared from material passing a No. 80 sieve that had been picked up on the surface of a droplet of mercury. The material was then transferred to a collodion-coated microscope grid by first softening the collodion coating in amyl acetate vapor and then pressing the grid against the clay-coated mercury droplet.

Transmission electron micrographs and electron diffraction patterns were made of specimens of kaolins H and E containing 6 percent lime. An electron microscope, operated at voltages of 80 and 100 kv., was used for the examinations.

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[1] Part of the results reported here were published in the Journal of the American Ceramic Society, 49(7), 1966, pp. 364-366.

[2] The italic numbers in parentheses identify the references listed on page 49.

[3] Sid Ahmed El Jack and James K. Mitchell, *Electron Microscope Studies of Interactions Between Hydrating Portland Cement and Soil Minerals.* Presented at the 44th Annual Highway Research Board Meeting, Washington, D.C., January 1965.

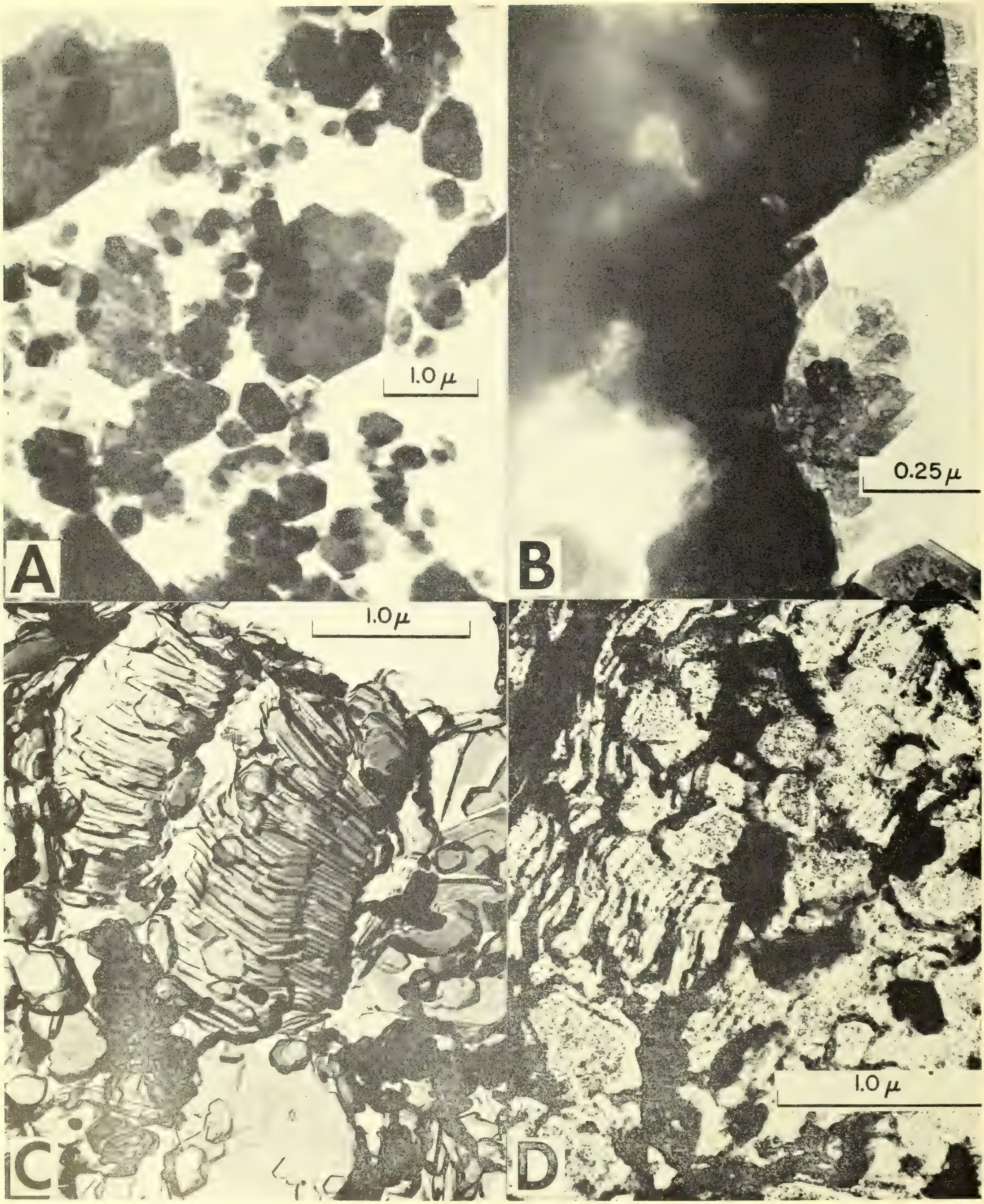

Figure 1.—Electron micrographs of (A) untreated kaolin E, (B) kaolin E plus 6 percent lime, cured 15 days, (C) replica of fracture surface of untreated kaolin E, and (D) replica of fracture surface of kaolin E plus 6 percent lime, cured 15 days.

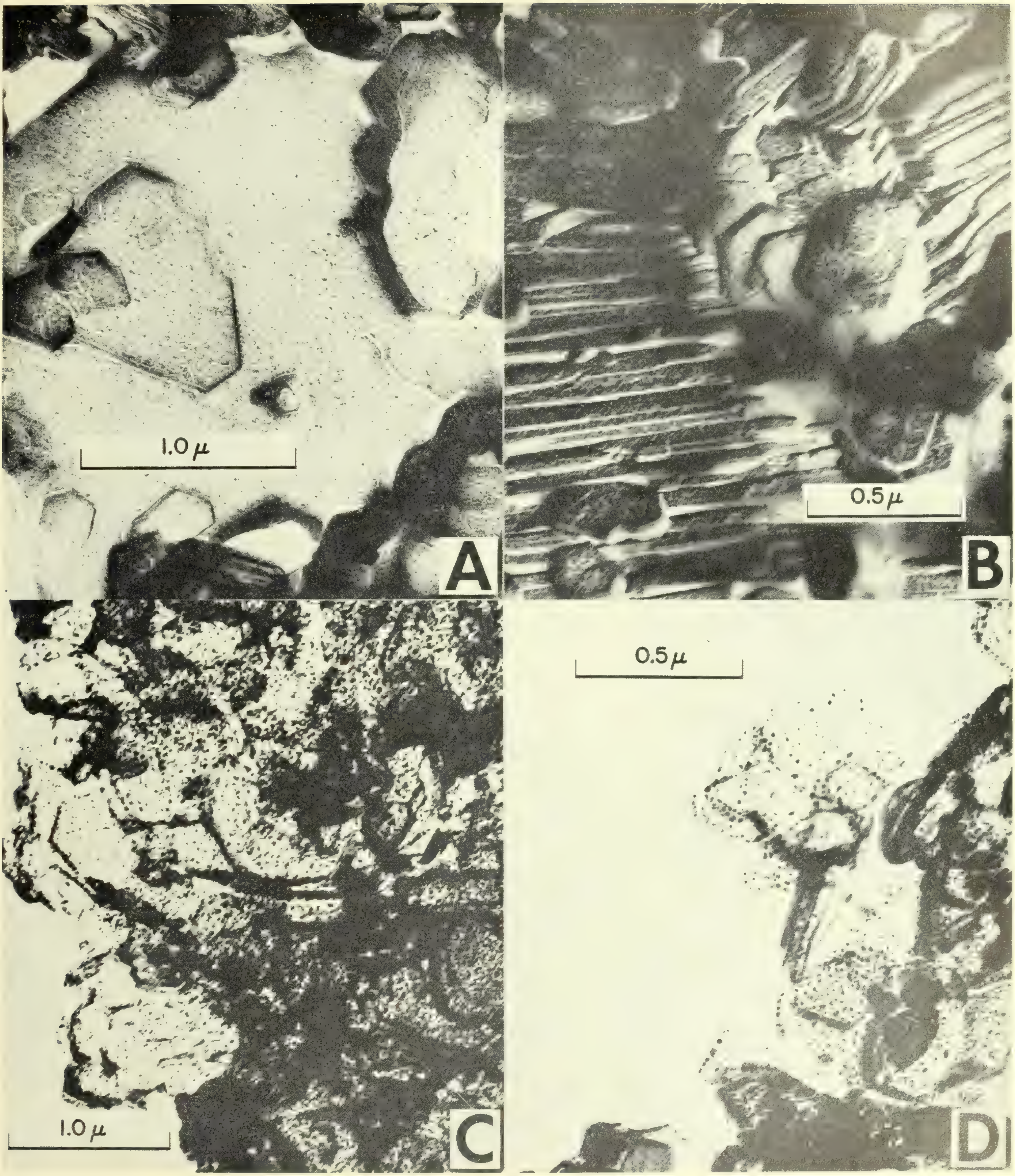

Figure 2.—Electron micrographs of (A) and (B) replicas of fracture surfaces of kaolin E (0.8 percent lime, cured 3 days), and (C) and (D) replicas of fracture surfaces of kaolin E (6 percent lime, cured 15 days).

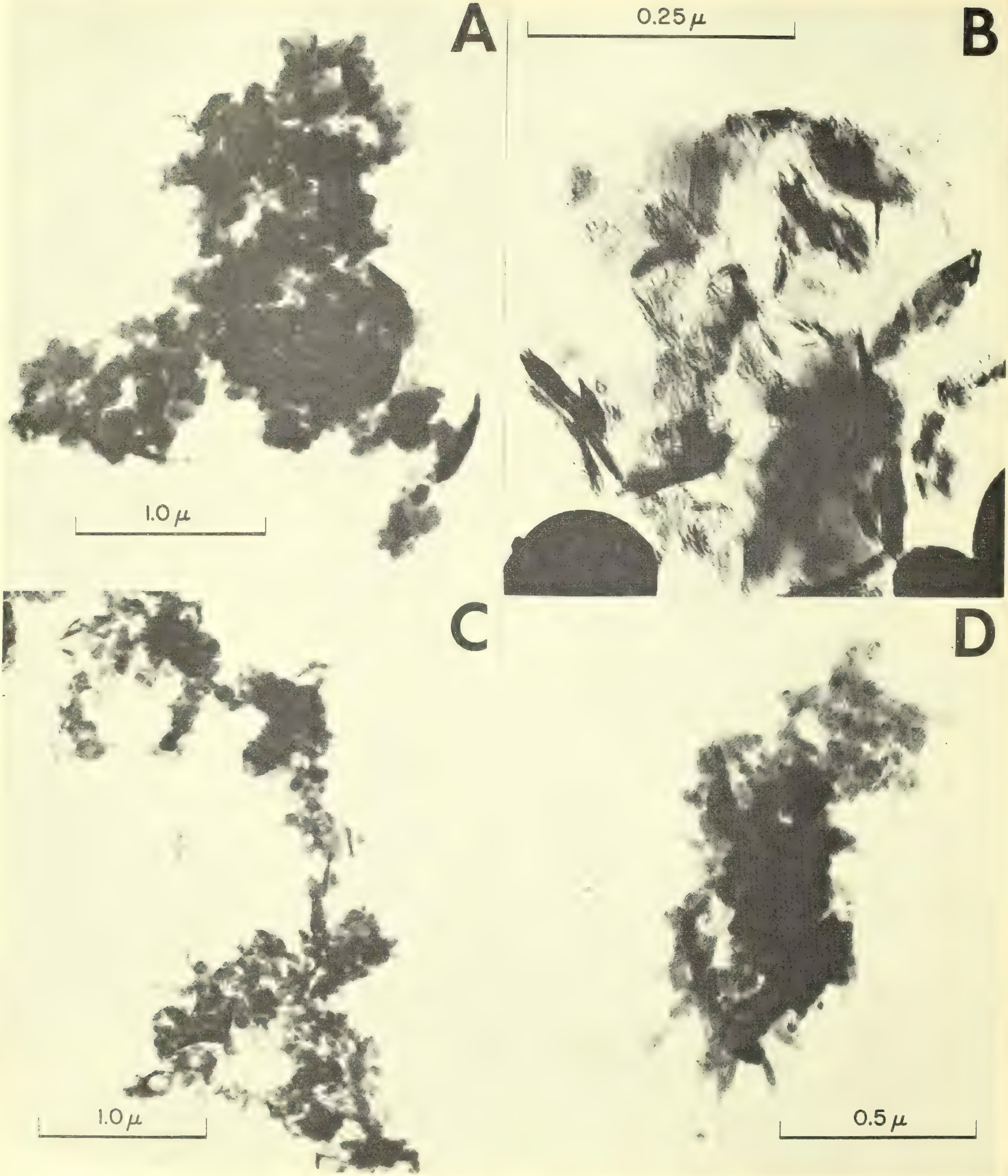

Figure 3.—Electron micrographs of (A) and (B) reaction product, kaolin E plus 6 percent lime, cured 15 days, and (C) and (D) reaction product, kaolin H plus 6 percent lime, cured 15 days.

Results and Discussion

Electron micrographs of morphological and textural features of unreacted clay, of clay reacted with lime, and of reaction products are shown in figures 1–3.

Hexagonal or pseudo-hexagonal shaped particles, typical for untreated kaolin E, are shown in figure 1(A). Figure 1(B), a photograph of kaolin E treated with 6 percent lime shows kaolinite particles severely eroded and pitted by the attack of the lime.

Photographs of replicas made from fracture surfaces of untreated and lime-treated kaolin E are shown in figures 1(C) and 1(D). Figure 1(C), in which several kaolinite plates and books are shown, is typical of the untreated kaolin. The plate and edge surfaces have a fine granular texture, but no surface deterioration is indicated. On the other hand, when the clay has been treated with lime, figure 1(D), basal surfaces and particle edges are severely eroded and many particles have lost their sharp, hexagonal outline. The opaque objects in figures 1(C) and 1(D) are from material that did not dissolve in the hydrofluoric acid used to remove the clay from the replica.

Figure 2(A) shows the fine structure of basal surfaces of kaolin E particles treated with 0.8 percent lime and cured for 3 days. Small hexagonal plates seem to be forming from larger plates as a result of surface attack. Progressive deterioration may result in the development of cleavages and fissures perpendicular to the basal surface and in the fragmentation of larger particles. Similar deterioration was observed by Oberlin and Tchoubar (*5*) in their study of the breakdown of kaolinite particles during $MgSO_4$ treatment. The process noted in the present study may, in the initial stages, be physically analogous to the effect produced by $MgSO_4$. In the experiment reported here, however, attack of the particle basal and edge surfaces may also result in the loss of structural components (Si, Al) and in the formation of new compounds (calcium silicate and calcium aluminate hydrates), possibly by a through-solution mechanism.

Figure 2(B) is a replicated surface of a kaolin E specimen treated with 0.8 percent lime and cured for 3 days. This photograph shows the characteristic reticular structure noted for basal surfaces (fig. 2(A)), as well as similar features for edge surfaces, suggesting that the lime has had a considerable effect on the clay, attacking both basal and edge surfaces of particles.

Figures 2(C) and 2(D) are pictures of replicas of fracture surfaces prepared from kaolin E treated with 6 percent lime and cured for 15 days; they show further examples of the type of attack that occurs at relatively high lime concentrations. Pitting and eroding of particles is severe. The overall appearance of the surfaces suggests that nucleation of new phases is occurring. The nuclei are of the order of 200A in diameter and are similar in appearance to nuclei observed by Sloane (*3*), who studied compositionally analogous systems.

Transmission electron micrographs of reaction products in kaolin-lime-water systems are given in Figure 3. Figures 3(A) and 3(B) are photographs of reaction products of kaolin E treated with 6 percent lime and cured 15 days. Figures 3(C) and 3(D) are of reaction products of kaolin H treated with 6 percent lime and cured 15 days. The reaction products were commonly seen together with degraded kaolinite particles (fig. 1(B)). The reaction products assumed various shapes, including films, fibers, and laths. Phases of nondescript particle shapes were also common; such phases could represent material of very poor crystallinity, that is, amorphous or gel material.

The products of poor to intermediate crystallinity, when examined by electron diffraction, gave *d* spacings of 3.03, 1.84, and 1.52A, which correspond to the indices 220, 040, and 440, respectively, for calcium silicate hydrate phases.

Pseudo-fracture surfaces gave replicas with features typical of kaolinite, pictures of which were similar to those published by Stetson (*6*). Replicas of the lateral surfaces of cylindrical specimens gave pictures that had the appearance of replicas of kaolin-coated paper; photographs of this type have been published by Bates (*7*). In the replicas of both pseudo-fracture surfaces and lateral surfaces of cylindrical specimens, the kaolinite particles were very well oriented with basal plans parallel to the plane of the deposited film. The replicas of the pseudo-fracture surfaces showed slightly more contrast and a very fine granular texture.

Summary

Electron optical investigations were performed on fracture surfaces of strength-determination specimens of kaolin-lime-water and on the same specimens that had been powdered. Results of these studies showed the following:

- Reactions between clay and lime occur at early stages of curing and at relatively low levels of lime addition.
- Attack occurs on both particle prism and basal surfaces.
- Reaction products in the systems studied are poorly crystallized calcium silicate hydrates, which are assumed to be responsible for strength improvements in kaolin-lime-water mixtures.

ACKNOWLEDGMENTS

The authors gratefully acknowledge discussions with Sidney Diamond and E. B. Kinter, and the assistance of A. C. Bonanno in preparing photographs.

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Direct Rapid Ultraviolet Determination of Phthalic Anhydride and Isophthalic Acid in Oil-modified Alkyd Resins and Paints

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by LEONARD BEAN, Research Chemist, and BERNARD CHAIKEN, Principal Research Chemist, Materials Division

Present standard procedures for determining the phthalic anhydride or isophthalic acid content of alkyd resins and paints, modified by drying oil, require prior chemical separation involving lengthy procedures. In this article a more rapid method is described by which these constituents can be determined directly in alkyd-resin paint vehicles within 2 hours, if other absorbing materials are known to be absent. The method is based on quantitative ultraviolet absorbance measurements of chloroform solution of the pigment-free vehicles after the original volatile solvents are removed, under vacuum, in a rotating flask evaporator. No prior chemical separations are required. Qualitative means for distinguishing alkyds, based on orthophthalic, isophthalic or terephthalic acids are demonstrated with infrared and ultraviolet spectra. This method is recommended for manufacturing process control, and especially purchase acceptance testing of alkyd and oil-alkyd paints. The method is generally applicable when the alkyd component is limited to any one of the three phthalate isomers and is not modified by other aromatics, unsaturated dibasic acids, or tung oil. Many of the coatings used in highways meet these requirements.

Figure 1.—Grating infrared spectrophotometer.

Introduction

ALKYD RESIN vehicles are commonly used in highway traffic paints and in other protective coatings for highway metal structures. For these uses alkyds are usually of the glyceryl orthophthalate type modified with ordinary drying oils such as linseed or soya.

Present methods published by the American Society for Testing and Materials (ASTM), for the determination of the phthalic anhydride content of alkyd resin paint vehicles require preliminary chemical separations that are relatively lengthy. All of the standard methods require an initial saponification during a refluxing period that is followed by precipitation and washing of the phthalate component. Depending on the method used, these procedures are followed by re-solution and either further precipitation or spectrophotometric analysis (*1*, *2*, *3*).[1] A similar method for determining the isophthalic acid content of alkyd resins, now under consideration by ASTM (*4*), also requires prior saponification and other chemical manipulations and separations.

In some of the larger State highway departments, the number of alkyd-type paints analyzed each year may exceed 300. Consequently, the present methods of analysis require considerable time and effort.

A direct and simple instrumental procedure to determine the phthalic anhydride or isophthalic acid content of drying oil alkyd resin paints, without resorting to prior chemical separations, would be desirable and save considerable time. Shreve and Heether (*5*) noted that phthalic acid and its esters show strong absorptions in their ultraviolet spectra, and that in some types of simple mixtures it should be possible to determine the phthalate ester content by direct ultraviolet absorption measurements of the mixture in a suitable

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[1] The italic numbers in parentheses identify the references listed on page 58.

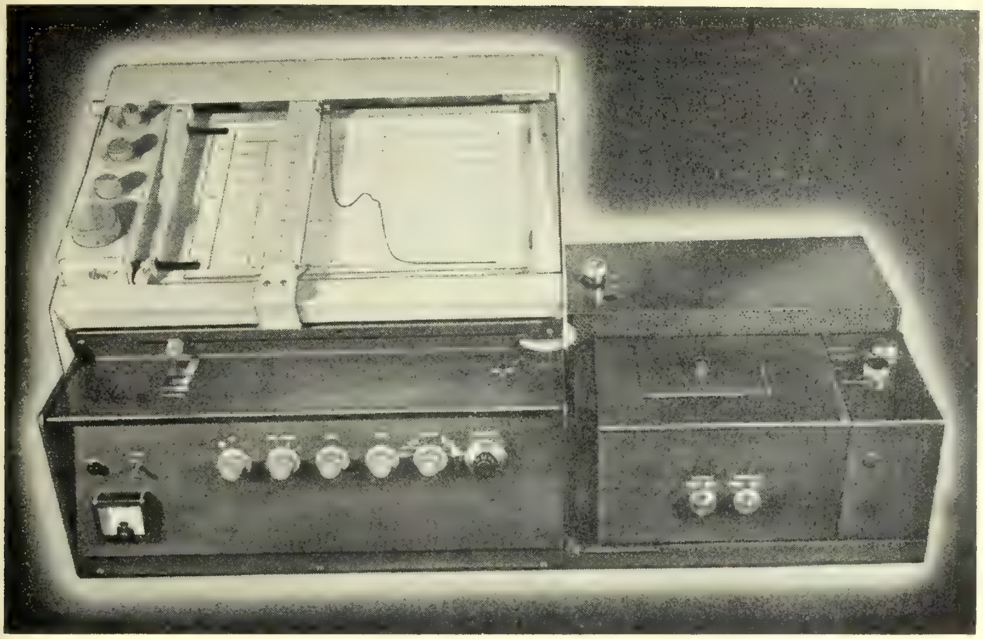

Figure 2.—Recording double-beam ultraviolet spectrophotometer.

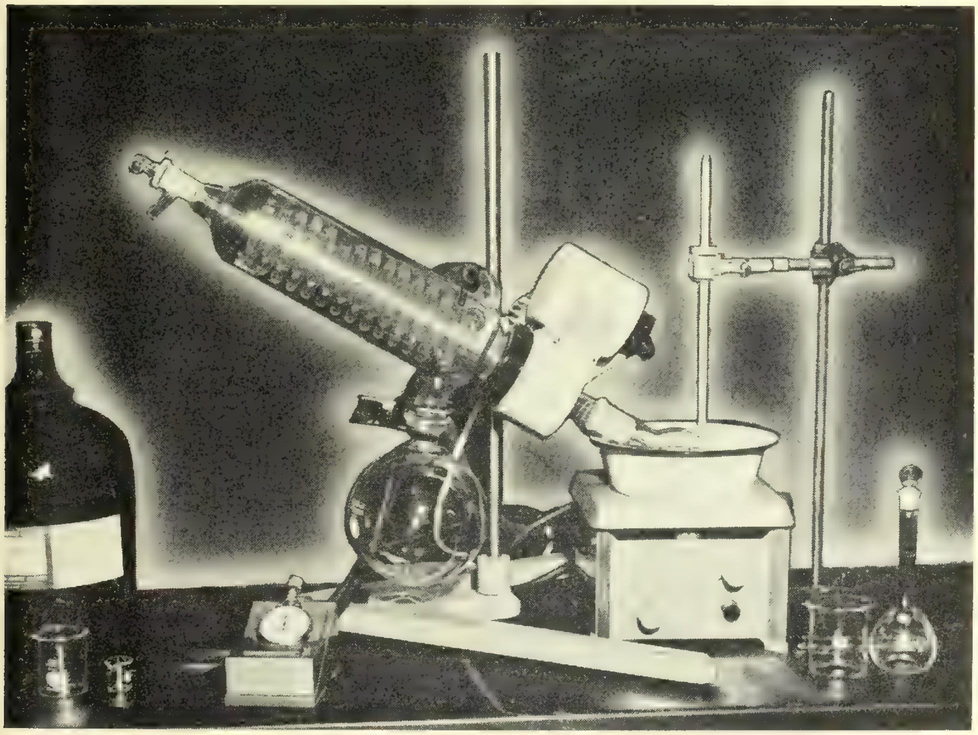

Figure 3.—Rotary-flask vacuum evaporator.

solvent. However, they reasoned that with paint vehicles and other types of coating materials, "such a procedure would be impractical because the organic solvents, oils, and other materials usually present would almost invariably interfere" Their reported work served as the basis of the present ASTM method for the spectrophotometric determination of phthalic anhydride in alkyd resins following preliminary chemical separations (*3*).

In a previous report (*6, 7*), a procedure for determining possible adulteration of traffic paint vehicles was described. During that investigation we observed that possibilities existed for applying similar rapid and direct ultraviolet techniques to facilitate the quantitative determination of the phthalic content of drying-oil modified alkyd resin coatings. It is believed that some of the limitations of a direct ultraviolet procedure for actual coatings, noted by Shreve and Heether, could perhaps be obviated. Accordingly, an investigation was conducted to determine what general identifying and quantitative information might be obtained by use of ultraviolet and infrared spectroscopy, without resorting to lengthy chemical separations. As a result the method described in this article was developed.

Apparatus

A grating infrared spectrophotometer (fig. 1) was used to scan the region from 4000 to 200 cm.$^{-1}$ while being purged with compressed air from which most of the CO_2 and H_2O was removed by passing the air through an *H_2O–CO_2 Absorber Unit.* The samples were cast films supported on cesium iodide windows. The instructions for operating the spectrophotometer are contained in the instruction manual (*8*) supplied by the manufacturer of the instrument.

A recording double-beam ultraviolet spectrophotometer (fig. 2), together with 1-cm. matched silica cells, was used to obtain spectral curves and measure the ultraviolet absorbance. A hydrogen discharge lamp was used as a light source for most of the measurements, but when it became necessary to make a replacement, a deuterium lamp was installed. A photomultiplier tube was used as the detector and the instrument was operated at a scanning rate of about 20 millimicrons (nanometers) per minute. The instructions for operating the spectrophotometer are contained in the instruction manual (9) provided by the manufacturer of the instrument.

A rotary flask vacuum evaporator (fig. 3) was used to remove the original volatile solvents from the paint vehicle.

Materials Studied

A number of manufacturers cooperated by supplying samples of drying-oil-modified alkyd resins prepared with different percentages of either phthalic anhydride or isophthalic acid. One alkyd made from terephthalic acid was also supplied. Additionally, the pigments were removed, by centrifugation, from four samples of drying-oil-modified alkyd type paints so that these vehicles could also be studied. A general description of these vehicles, as well as quantitative information on their composition, is given in table 1.

Many of the raw materials used in the manufacture of alkyd vehicles were examined. These included various polyols, such as glycerol, ethylene glycol, and pentaerythritol. Drying oils, such as various types of linseed and soybean oil, and one tung oil were also examined.

Qualitative Infrared Examination of Oil-Alkyd Resins

Tests were made to determine whether infrared analysis could be used to establish the nature of the phthalic acid present. Differences were easily noted among the infrared

Table 1.—Description of oil-modified alkyd resins used in study

Alkyd resin	Laboratory number	Polyol	Percent phthalic anhydride or phthalic acid
Orthophthalic type:			Percent phthalic anhydride [1]
Vehicle from oil-alkyd bridge paint primer	C 1929		7.8
Vehicle from oil-alkyd bridge paint primer	C 1836		7.8
Vehicle from oil-alkyd bridge paint primer	C 1908		14.5
Vehicle from oil-alkyd bridge paint primer	C 1837		15.0
Long linseed oil resin	C 1895	glycerol	23.0
Resin containing soybean oil	C 1858	glycerol and pentaerythritol.	28.2
Medium linseed oil resin	C 1892	glycerol	30.2
Medium linseed oil resin, 52% oil	C 1901	glycerol	32.6
Resin containing soybean oil	C 1860	pentaerythritol and ethylene glycol.	33.0
Resin containing 52% soybean oil	C 1817	glycerol	33.8
Short linseed oil resin, 48% oil	C 1902	glycerol	39.0
Soybean oil resin	C 1861	glycerol	41.7
Short linseed oil resin	C 1894	glycerol	41.6
Isophthalic acid type			Percent isophthalic acid
A long soya oil resin, 74% oil	C 1932		19
A long soya oil resin	C 1916		24
A long safflower oil resin, 64% oil	C 1913		27
A long safflower oil resin, 61% oil	C 1933		27
A medium soya resin, 59% oil acids	C 1930		28
A medium soya oil resin	C 1917		33
A linseed oil modified resin, 50% oil, 8.8% para tertiary butylbenzoic acid.	C 1912		33.9
A short soya oil resin, 40% oil acids	C 1931		38
Terephthalic acid type			Percent terephthalic acid:
A short soya oil resin, 40% oil, 14.7% benzoic acid (values furnished by manufacturer).	C 1945	glycerol	34.4

[1] The phthalic anhydride content was determined as described in ASTM Designation D 563-62, 1967 Book of ASTM Standards, Part 20.

[2] No information available concerning the polyol. Information on the isophthalic acid and oil contents was supplied by the manufacturer.

patterns of oil-modified alkyd resins made from either phthalic anhydride, isophthalic acid, or terephthalic acid, when approximately 15 percent or more of the phthalic constituent was present. White has observed (*10*) that *o*-phthalic esters have a characteristically sharp doublet (sometimes referred to as *rabbit ears*) in the vicinity of 1600 cm.$^{-1}$, which is probably due to aromatic skeletal in-plane C=C vibrations. This absorption enables polymer chemists to spot *o*-phthalic alkyd resins at a glance.

In the infrared patterns of eleven *o*-phthalic resins examined in this study, the right *ear*, or band, at 1570 to 1580 cm.$^{-1}$, was always somewhat longer than the *ear* at 1595 cm.$^{-1}$, as shown in figure 4. In the infrared patterns of the two remaining vehicles, which contained less than 8 percent phthalic anhydride, the *rabbit ears* were not evident. In fact, the patterns of these two vehicles resembled that of linseed oil, the predominant constituent. On the other hand, in the patterns of all eight isophthalic resins examined, the doublet peaks shifted to approximately 1608 and 1585 cm.$^{-1}$, with the right *ear*, at 1585 cm.$^{-1}$, being much shorter than the left one at 1608 cm.$^{-1}$. This is illustrated in figure 5. In fact, in one isophthalic resin (fig. 6) the right *ear* had practically disappeared. In the pattern of the only terephthalic alkyd resin examined, the doublet peaks were at approximately 1600 and 1575 cm.$^{-1}$, with the right *ear* being somewhat shorter, as shown in figure 7. Thus, the exact wavelength positions and relative depths of the doublet peaks near 1600 cm.$^{-1}$, should easily indicate whether an orthophthalic, isophthalic, or terephthalic alkyd was the dominant material present in the oil modified resin. These criteria are quite useful in a qualitative evaluation of resins of unknown composition.

The region of 650 cm.$^{-1}$ was also investigated as a possibility for distinguishing between the types of phthalic acid present in an oil-alkyd resin. This band is attributed to the degenerate bending vibration of the ester group. It has been reported (*11*) that oil-modified alkyd resins, based on isophthalic acid, normally show a pair of bands near 650 cm.$^{-1}$, whereas *o*-phthalates have a single and sharp band. The spectra of most of the alkyds used in this study, as exemplified by figures 4, 5, and 6, failed to show these distinguishing characteristics. In fact, the bulk of both the orthophthalic and isophthalic alkyds showed similar and only single sharp bands at 650 cm.$^{-1}$. As shown in figure 7 the pattern of terephthalic resin examined showed no band at 650 cm.$^{-1}$.

Applicability of Ultraviolet Spectroscopy

Measurements were made, without prior chemical separations, to determine whether a correlation could be found between ultraviolet absorbance at definite wavelengths and the phthalic anhydride, or isophthalic acid, content of oil modified alkyd resins.

In general, the procedure used was that described in previous reports by the authors (*6, 7*). The volatile solvents were removed at 56–60° C. from a weighed sample of the vehicle, using a rotating flask vacuum evaporator for 20 minutes. Then the remaining solid material was dissolved promptly in a solvent, diluted to volume, and aliquots were taken to provide suitable concentrations for ultra-

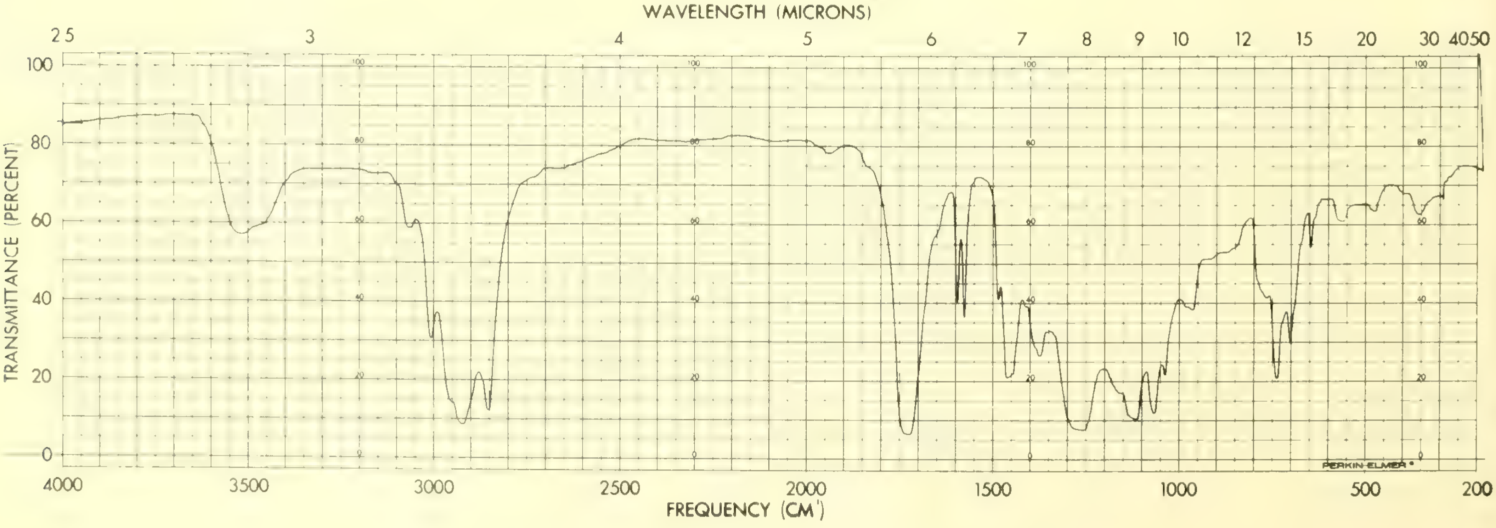

Figure 4.—Infrared spectrum of medium soya oil o-phthalic alkyd resin.

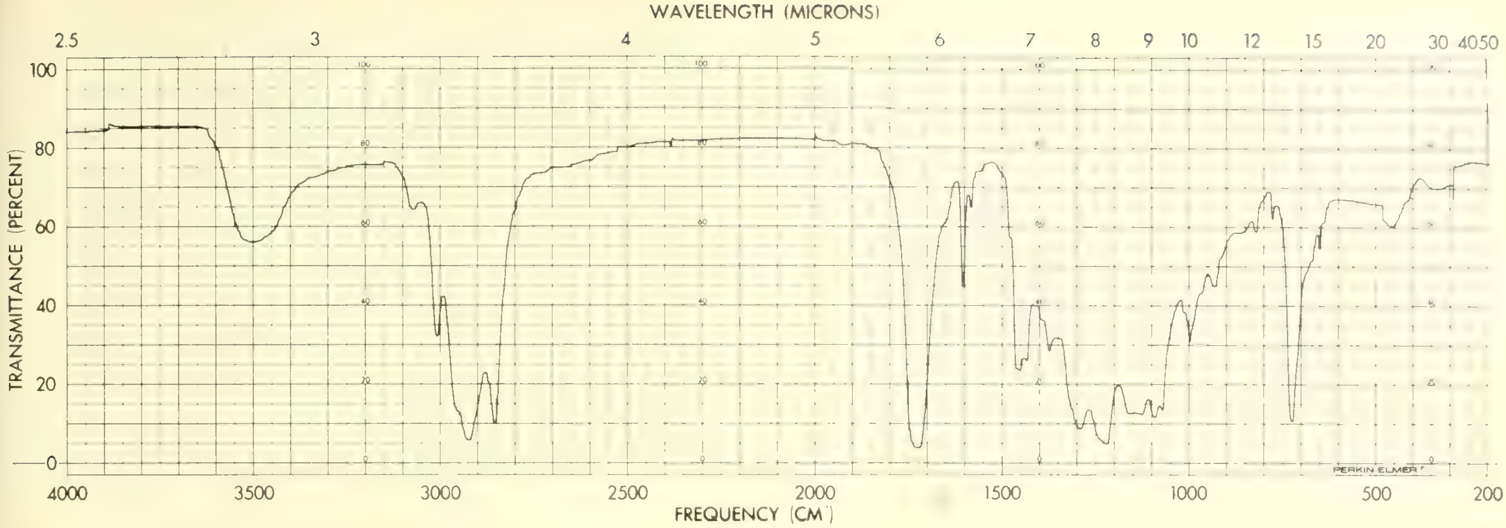

Figure 5.—Infrared spectrum of medium soya oil isophthalic alkyd resin.

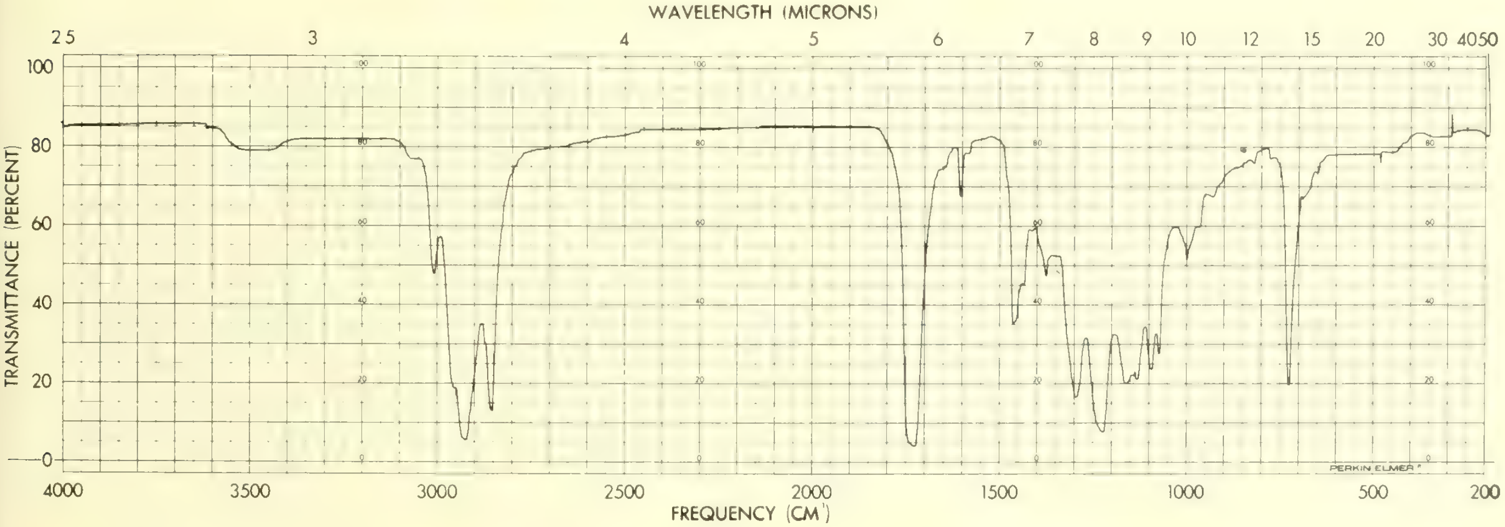

Figure 6.—Infrared spectrum of long soya oil isophthalic alkyd resin.

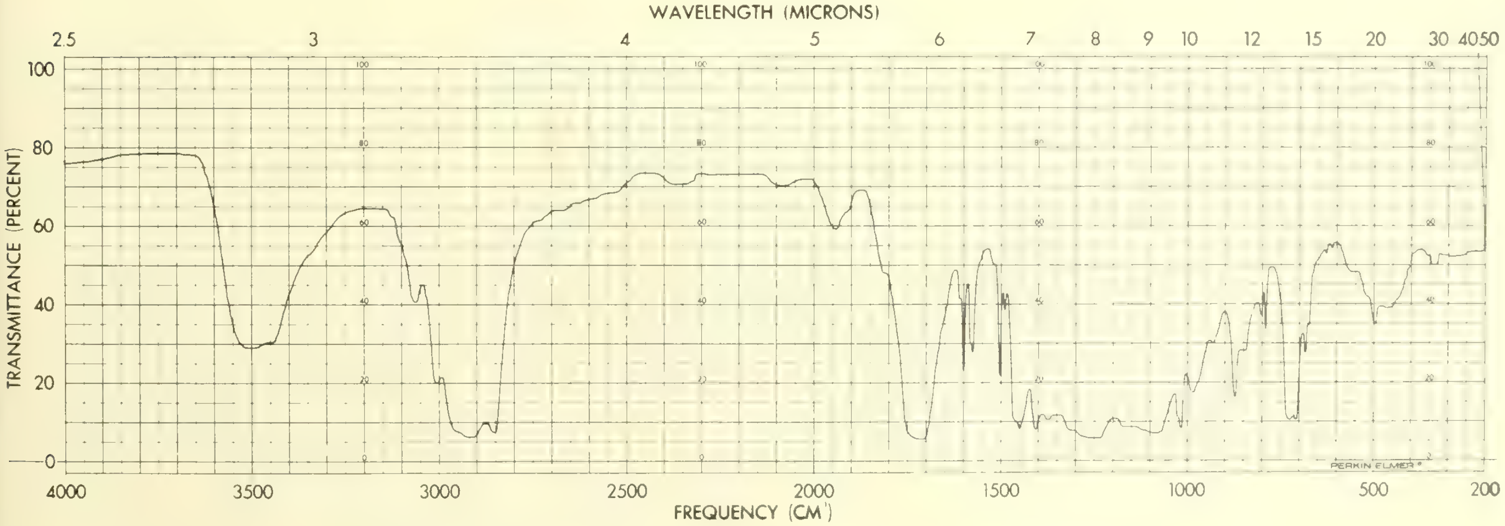

Figure 7.—Infrared spectrum of short soya oil terephthalic alkyd resin.

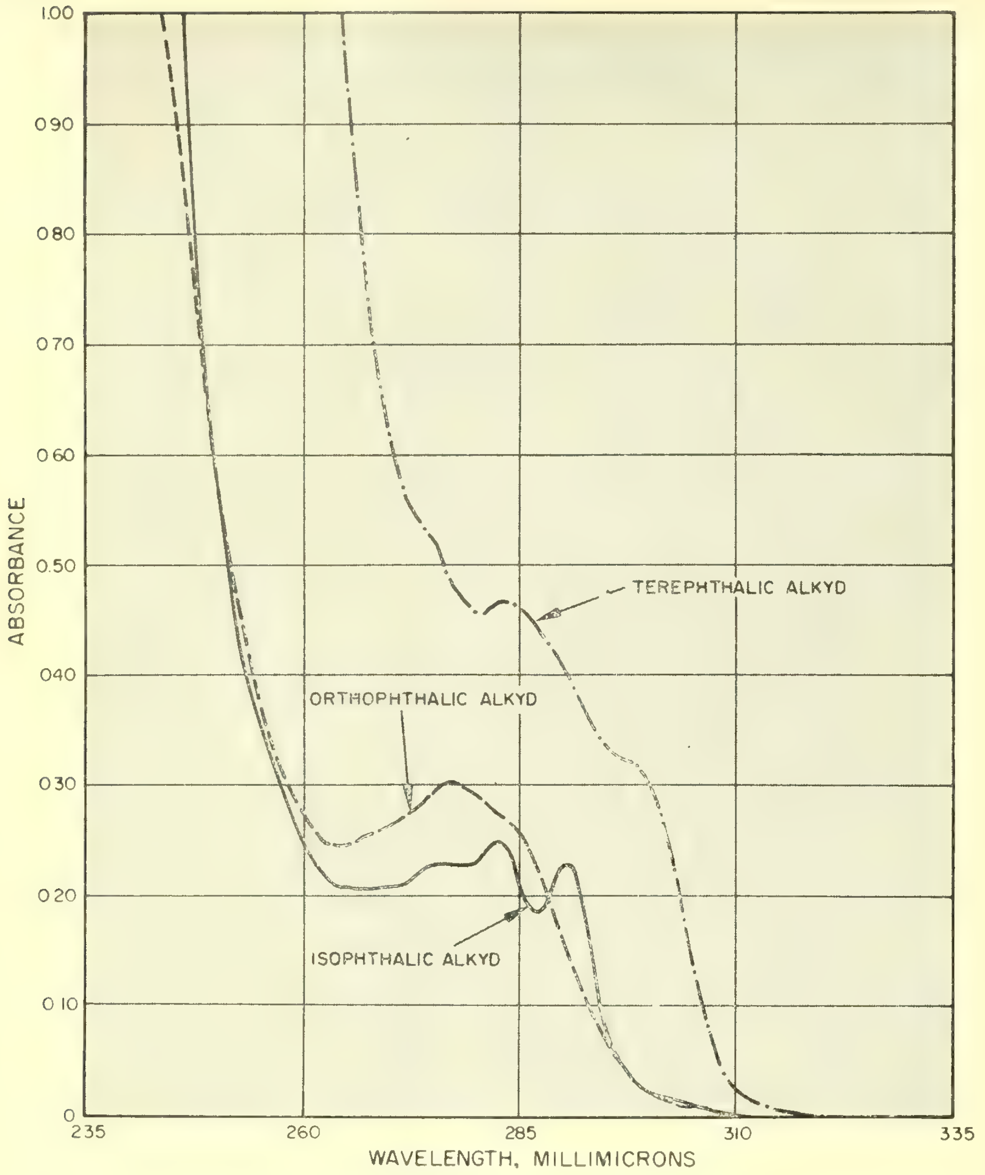

Figure 8.—Characteristic ultraviolet spectral curves of the three phthalate varieties of oil modified alkyds, 0.1 g/l concentration in $CHCl_3$.

violet absorbance measurements. For most of the measurements reported previously (*6*, *7*), cyclohexane was used as the solvent. This was fairly satisfactory as a solvent and could be used throughout the range of 220 to 360 millimicrons. Chloroform was used as a solvent for the oil alkyd resins studied in this investigation. Although this solvent is applicable only in the range from 240 to 360 millimicrons, it had, for our purposes, solvent properties superior to cyclohexane. In these experiments, solutions of the oil modified alkyd resins in chloroform remained clear indefinitely, whereas concentrated cyclohexane solutions of the resins became cloudy on standing.

Figure 8 shows typical ultraviolet absorption curves for solutions containing 0.1g/l of oil modified orthophthalic, isophthalic, and terephthalic acid alkyd resins in chloroform. All three materials produced characteristically different spectra. The orthophthalic resin was characterized by a peak at 276 millimicrons and a slight shoulder at about 284 millimicrons. These characteristics can be recognized clearly even in oil-resin mixtures containing less than 8 percent phthalic anhydride. In contrast, infrared spectra failed to indicate the phthalate characteristics at this low content of phthalic anhydride. As shown, three peaks at 291, 282, and 276 millimicrons are typical of an isophthalic resin. In some of the other isophthalics studied, the peak at 276 millimicrons is quite weak and is evident only as a shoulder. Terephthalic resin is distinguished by a peak at 283 millimicrons, a shoulder at about 298 millimicrons, and another slight shoulder at about 274 millimicrons. Similar distinguishing ultraviolet characteristics were reported by Kappelmeier (*12*) for the pure potassium salts of the various phthalic acids after they had been separated chemically from the parent oil modified alkyd resin. Thus, as a supplement to infrared analysis, it should be possible to determine qualitatively whether an alkyd resin was made from orthophthalic, isophthalic, or terephthalic acid, after the volatile solvents have been removed, by observing the pattern of the ultraviolet absorbance curve of a paint vehicle without the necessity of prior chemical separations. Based on these results, the quantitative possibilities of a direct ultraviolet procedure appeared to have merit and were therefore fully investigated, as reported in the following paragraphs.

Possible Interferences in a Direct Ultraviolet Procedure

Other than the phthalic acids or phthalic anhydride, many materials used in the manufacture of alkyd resins will absorb ultraviolet radiation (*5*). Therefore, measurements were made on such materials to determine whether their ultraviolet absorbances at the wavelengths of interest might interfere with a quantitative evaluation of the content of phthalic anhydride or isophthalic acid.

Possible interfering materials to be considered include the following: the solvents; the polyols (polyhydric alcohols)—glycol, glycerol, and pentaerythritol; dibasic acids other than phthalates, such as, saturated types—succinic, adipic and sebacic, and monounsaturated types—itaconic, maleic, and fumaric; and the various drying oils.

Interference from the commercially used solvents, notably the aromatics, can be immediately dismissed because the contemplated method involved the complete removal of the original solvent. The effect of the dibasic saturated acids can also be disregarded, as they exhibit comparatively insignificant absorptivity in the ultraviolet region under consideration (*5*, *12*). Monounsaturated dibasic acids—itaconic, maleic, and fumaric—can possibly interfere (*5*, *12*, *13*), but they are rarely used in highway coatings, so these potential effects were neglected in the investigation.

Consequently, this phase of the investigation was limited to studying the possible interfering effects of various drying oils and polyhydric alcohols used in alkyd manufacture. Ultraviolet absorbance measurements were made on appropriate solutions of various drying oils including tung and fish oil; and ethylene glycol, pentaerythritol, and glycerol. Calculations were then made to determine the absorptivity of these dissolved materials at the wavelengths of interest. From these calculations it appeared that none of these materials, except tung oil, should cause any measurable interference. For example, fish oil, even if present to an extent of 50 percent in an alkyd resin, will merely contribute an absorbance value of 0.009 at a total solids concentration of 0.1 g/l, and therefore affect the results only slightly. Tung oil, however, absorbs strongly. An absorbance value of 0.95 (equivalent to an absorptivity of 95) was obtained at 276 millimicrons for a 0.01 g/l solution in chloroform. The spectral interference of tung oil is produced by its eleostearic acid component, which has a triple conjugated bond structure and is therefore, highly absorptive at 276 millimicrons (*14*, *15*).

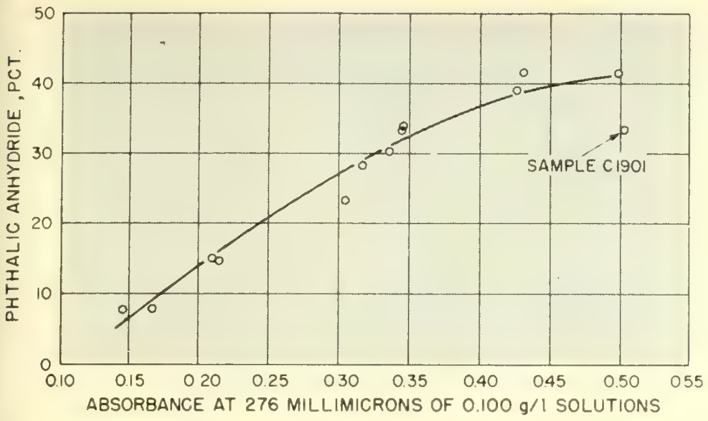

Figure 9.—Relation between phthalic anhydride content and ultraviolet absorbance of oil modified o-phthalic alkyd at 276 millimicrons, 0.1 g/l concentration in $CHCl_3$.

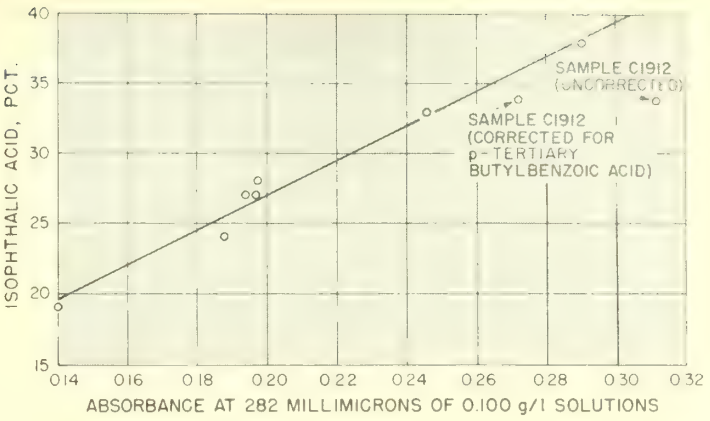

Figure 10.—Relation between isophthalic acid content and ultraviolet absorbance of oil modified isophthalic alkyd at 282 millimicrons, 0.1 g/l concentration in $CHCl_3$.

The type of conjugation may be determined by the location of the characteristic band maximum. In round numbers, a band near 230 millimicrons indicates diene conjugation, at 270 millimicrons—triene conjugation, and at 310 millimicrons a system of four conjugated bonds (*16*). Clearly then, the presence of any significant amounts of tung oil, which also has a high maximum absorbance at 276 millimicrons, would vitiate any attempts to correlate the amount of phthalic anhydride present with direct absorbance measurements made at 276 millimicrons.

Other workers have also pointed out [2] that other constituents of special alkyd resins could cause high absorption at 276 millimicrons. Among these are polystyrene, benzoic acid, vinyl toluene, epoxies, phenolic resins, coumarone-indene, chlorinated polyphenyl, and aromatic silicones. Such high absorption would occur when any aromatic adjunct is present in an alkyd, as the conjugated aromatic structure is highly absorptive in the ultraviolet region. Fortunately, such materials are not customarily incorporated in oil modified alkyds that are specified to be entirely of the *o*-phthalic or isophthalic type.

Quantitative Ultraviolet Measurements

Using the ultraviolet procedure described in detail in previous reports (*6, 7*), and given in condensed form under *Recommended Test Procedure*, quantitative absorbance measurements were made on the oil modified *o*-phthalic and isophthalic alkyd resins and paints listed in table 1. Where necessary, the pigment was removed by centrifuging and the original solvent eliminated with the rotary vacuum evaporator. Chloroform solutions of the vacuum dried total solids were then prepared to contain exactly 0.100 g/l. Care was exercised to avoid undue exposure to the atmosphere.

Figure 9 shows the relation between the chemically determined phthalic anhydride content of oil modified *o*-phthalic alkyd resins and the absorbance value obtained at 276 millimicrons. In each case, the phthalic anhydride content plotted represented an average of two determinations. All but one point showed a fairly linear relationship throughout most of the range of values. These vehicles had been manufactured by seven different companies. The polyol constituent of the resins was either glycerol, ethylene glycol and pentaerythritol, or glycerol and pentaerythritol. Most of the resins contained glycerol. Linseed or soybean oil was present as the drying oil modification in each sample.

The plotted value for one sample, C1901, is far from the line drawn through the other points. Assistance in explaining the behavior of this sample was obtained from the Coating and Chemical Laboratory, Aberdeen Proving Grounds (A.P.G.). Their examination of the material showed that the high absorbance was caused by tung oil, present as an adjunct to the drying oil. The identification of the tung oil was made by extracting the oil acids following saponification and removal of unsaponifiable matter. The solvent-free oil acids were then dissolved in methanol and ultraviolet absorbance measurements made. The oil acids showed moderate absorption bands at 259, 269, and 280 millimicrons, the 269 band being the strongest. These bands are characteristic of tung oil eleostearic acids.[2] The A.P.G.'s findings were later corroborated in Public Roads' laboratory by a similar separation of oil acids and confirmation of the presence of bands of tung oil eleostearic acids.

A number of qualitative tests are recommended for the detection of the presence of tung oil in a mixture (*17*). Of these, the Wan Test gave a positive indication of tung oil.

Figure 10 shows the relation between isophthalic acid content, as provided by the producers, of drying oil modified isophthalic alkyd resins and absorbance at 282 millimicrons. All but one sample, C1912, indicate a fairly linear plot. The absorbance and peculiar behavior of sample C1912 are discussed in the succeeding paragraphs.

The ultraviolet absorbance of isophthalic alkyd solutions showed no evidence of changing with time or exposure to light, in contrast to the behavior of phthalic anhydride alkyds reported in an earlier paper (*6, 7*). Even an 89-hour exposure to an ultraviolet lamp caused no appreciable absorbance change in the isophthalic alkyds. Apparently, no special precautions must be observed when making absorbance measurements on isophthalic alkyds. A possible explanation for this difference in exposure behavior between *o*-phthalic and isophthalic acid is that the presence of minute amounts of quinones produces progressive changes in the absorbance of the *o*-phthalic resins. It has been reported that 1, 4-naphthoquinone may occur as an impurity in the manufacture of phthalic anhydride, and that although it absorbs ultraviolet radiation quite strongly, it is progressively oxidized and, thereby, its effect is reduced with time (*18*). Isophthalic acids are not known to contain such quinone impurities. This appears to account for the difference in exposure behavior.

Anomalous Absorbance of Sample C1912

One isophthalic alkyd, sample C1912, gave an unusually high absorbance at 282 millimicrons, as shown in figure 10. Information obtained from the manufacturer stated that 8.8 percent para tertiary butylbenzoic acid was used in the formulation of that resin. Ultraviolet absorbance curves were made of a *practical* grade of para tertiary butylbenzoic acid dissolved in chloroform 0.100 g/l. At 282 millimicrons the absorptivity was 4.27. When the absorbance of sample C1912 is corrected for the absorbance contributed by the presence of 8.8 percent para tertiary butylbenzoic acid, a corrected point, much nearer to the line drawn through the other points, is shown in figure 10.

A close examination of the ultraviolet absorption pattern for sample C1912 shows a

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[2] Private communications with M. H. Swann and G. G. Esposito, Coating and Chemical Laboratory, Aberdeen Proving Ground, U.S. Department of the Army, Aberdeen, Md.

Table 2.—Repeatability of absorbance measurements made on isophthalic resin sample C1917 [1]

Replicate runs	Absorbance value at 282 millimicrons
Determination number:	
1	0.243
2	0.250
3	0.246
4	0.246
5	0.244
Average	0.246
Range	0.007

[1] In A. C. S. reagent grade chloroform at concentration of 0.1 g/l.

slight shoulder at about 302 millimicrons (see fig. 11). This shoulder was not present in the patterns of any of the other isophthalic alkyds examined, as exemplified by figure *8*. The isophthalic acid was isolated from this alkyd, sample C1912, by saponification and acidification (*4*), and an ultraviolet absorption curve was made on a methanol solution of it. The anomalous shoulder was still present, but in the free acid form it had shifted to about 298 millimicrons (see fig. 12). In this region, there was no comparable absorptivity shown in a spectral curve for a sample of para tertiary butylbenzoic acid. Thus, some additional unidentified interfering material appeared to be present.

Commercial isophthalic acid is known to contain some terephthalic acid. This is especially true if the isophthalic acid has been prepared from a mixture of meta and para xylene. To explore this possibility, an ultraviolet absorbance curve was made of a methanol solution containing 0.100 g/l of commercial terephthalic acid. At 302 millimicrons, this material had an absorptivity of about 1.20. Accordingly, the presence of some terephthalic acid could easily account for the shoulder that was present in the ultraviolet pattern of sample C1912. As can be seen from figure 8, such an impurity could produce a shoulder at about 302 millimicrons. If the chemical method of isolating the free isophthalic acid from this resin (*4*) failed to provide a perfect separation of isophthalic and terephthalic acids, the persistence of the shoulder in the ultraviolet pattern of the isolated free acids is thus explained. At 282 millimicrons, the absorptivity of the terephthalic acid solution was about 4.60. Therefore, any terephthalic acid present, in addition to isophthalic acid and para tertiary butylbenzoic acid, would contribute to the anomalous high absorbance value of sample C1912.

As further verification of the presence of these interferences, the infrared spectrum of the anomalous resin is shown in figure 13. Four sharp bands are noted by asterisks at 771, 1018, 1408, and 1500 cm.$^{-1}$. These bands are essentially absent in the other isophthalic alkyds, as exemplified by figures 5 and 6.

From an examination of infrared patterns of para tertiary butylbenzoic acid, terephthalic alkyd (fig. 7), and terephthalic acid, it was noted that these same four bands were present in each. Thus the anomalous high absorbance value for this resin is clearly explained by the presence of two other ultraviolet absorbing species in addition to isophthalic acid. These findings serve to illustrate the value of qualitative infrared and ultraviolet examinations preliminary to quantitative ultraviolet analysis.

Repeatability of Quantitative Ultraviolet Measurements

General repeatability data were not obtained in this study; but some indication of the precision of the method was obtained by making five independent measurements on freshly prepared solutions from one isophthalic resin over a 3-day period. Measurements were made at 282 millimicrons on 0.1 g/l chloroform solutions following the procedure described above. The measured absorbance values are given in table 2. The calculated range in absorbance of 0.007 represents a range in percentage of isophthalic acid of 0.9 when read from the curve shown in figure 10. Thus, at this level of resin content, the repeatability appears quite satisfactory.

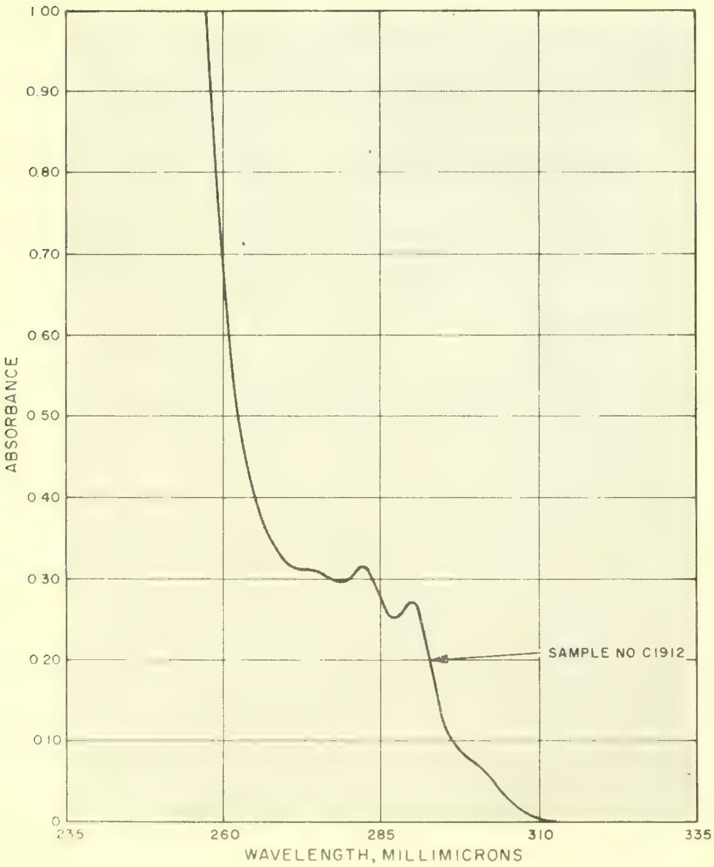

Figure 11.—Ultraviolet spectrum of oil modified isophthalic alkyd—sample C1912—that gave anomalous results, 0.1 g/l in $CHCl_3$.

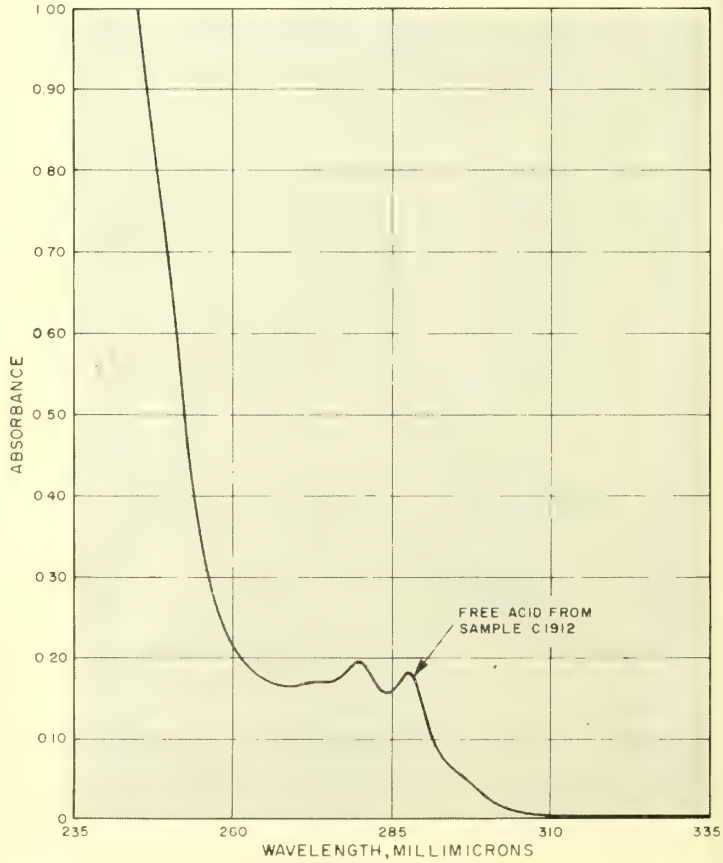

Figure 12.—Ultraviolet spectrum of isophthalic acid isolated from parent alkyd material, C1912, shown in figure 11 (in CH_3OH).

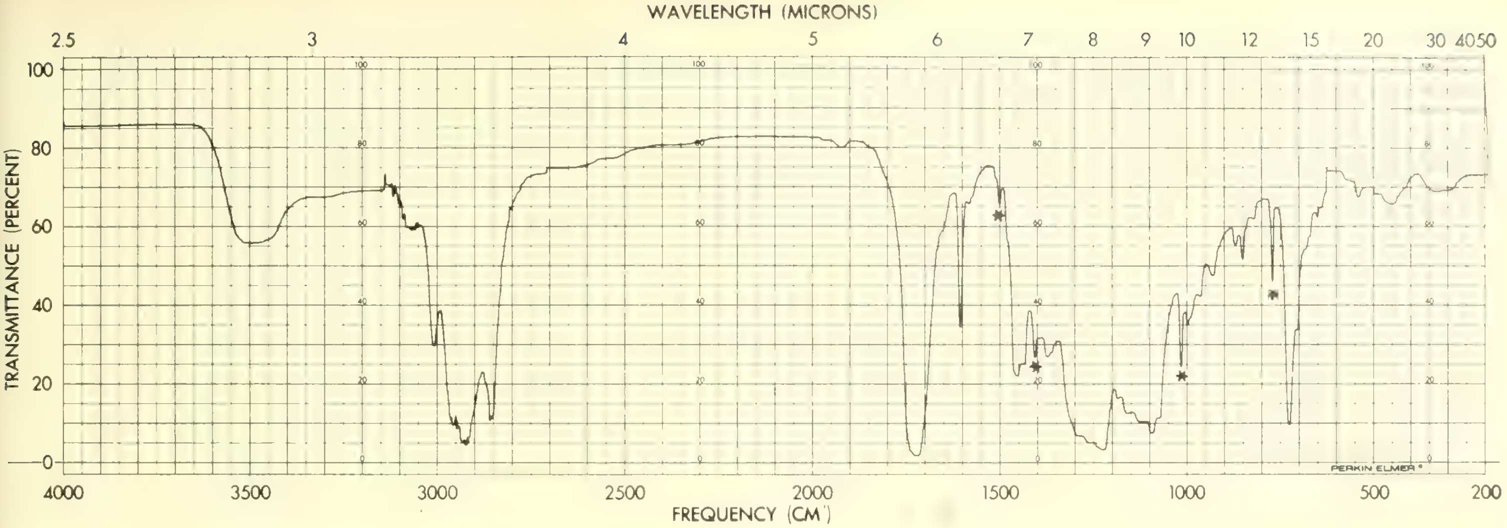

Figure 13.—Infrared spectrum of anomalous oil modified isophthalic alkyd, C1912. The four asterisks indicate bands that are essentially absent from spectra of the other isophthalic alkyds studied.

Repeatability measurements of the ultraviolet absorbance values were not made for the phthalic anhydride content of the oil modified orthophthalic alkyds used. Such measurements were available from a previous investigation (*6, 7*). Based on eight replicate measurements made in that study, it would appear that the repeatability of the phthalic anhydride content determination by the recommended procedure would be of an acceptable level, but somewhat less precise than for the isophthalic acid determination. The cause for this difference is explained by the previous discussion regarding quinone impurities and exposure changes.

Use of Cesium Iodide (CsI) Windows

Cesium iodide windows for infrared measurements throughout the range of 4000 to 200 cm.$^{-1}$ were used in the investigation. Cesium iodide is a very soft material that is easily scratched, bent, and corroded by water. A single window (or plate) was used to obtain the infrared spectra of alkyd resins. A few drops of chloroform solution of the alkyd were placed on the window and dried at 60° C. in a vacuum oven—a practice that was much simpler than using two windows. When the resin was sandwiched between two windows, the windows could not be separated without injuring them.

On one occasion a CsI window became warped while it was drying in the vacuum oven at 60° C. To correct the warping, the window was placed between two steel plates with filter paper separating the steel and the window. Warping was eliminated after the assembly was heated overnight, under vacuum, at 60° C.

Frequent polishing and occasional grinding of the CsI windows were determined to be necessary. In fact, as a routine matter, the CsI window was nearly always polished after each film had been removed with solvents. Thus, the transmittance of the window was satisfactory for use on the next sample to be examined, and grinding was seldom required. The following polishing technique was used [3]: A piece of chamois was stretched over the surface of a square glass plate. This was moistened with reagent grade 2-propanol and the CsI window polished with figure-eight motions. Plastic gloves were worn during polishing and grinding operations. Progress of the polishing operation was easily checked by placing the window in the sample beam of the spectrophotometer. A window that gave a transmittance of 83 to 86 percent at 4000 cm.$^{-1}$ was very satisfactory. This was approximately the transmittance of the windows when new.

A convenient method for grinding the windows when they became badly etched was to use a piece of number 600 waterproof silicon carbide abrasive paper placed on a flat surface. Then they were polished as described above. Other suitable procedures for grinding and polishing CsI windows have been published (*19, 20*).

Summary and Conclusions

The study reported here resulted in a demonstration of the usefulness of infrared and ultraviolet spectroscopy for the rapid qualitative and quantitative characterization of oil-modified alkyd resin paint vehicles that have not previously been separated chemically. A preliminary infrared examination will generally indicate whether a phthalate type alkyd is present and also provide evidence as to type—orthophthalate, isophthalate or terephthalate, or even mixtures of two of these. The type can also be substantiated by ultraviolet analysis. When the type is known, a suitable calibration curve can be used to determine the phthalic anhydride or isophthalic acid content by quantitative ultraviolet absorbance measurements using the procedure described under *Recommended Test Procedure*. The procedure requires no prior chemical separation; it requires only pigment separation, if necessary, and the removal of the original solvent under vacuum. Such determinations take about 2 hours, whereas about 5 hours are required to perform the necessary chemical separations and determine phthalic anhydride or isophthalic acid by ASTM and other available gravimetric and spectrophotometric methods. The more rapid procedure should be of considerable value to highway and other laboratories that conduct tests on a large number of alkyd resins and alkyd paint samples.

In the ultraviolet determination of isophthalic acid, a repeatability range for five determinations was obtained that represented 0.9 percent isophthalic acid. In connection with a previous, related investigation by the authors, it can be shown that the range for eight determinations represented 2.0 percent phthalic anhydride.

Reliance on the results of the test must be based on a knowledge of the absence of interfering substances. When tung oil is used as part or all of the drying oil modification, considerable error may occur. Such interfering substances may be detected by appropriate spot tests. Ordinarily, it would not be expected that the manufacturer would substitute tung oil when more economical drying oil modifications are specified for an alkyd.

The presence of other highly absorbing materials, such as terephthalic acid or para tertiary butylbenzoic acid, can also cause errors in the determination of isophthalic acid or phthalic anhydride by the method described. Use of the rapid method is not recommended when such adjuncts are known to be present or when qualitative infrared or ultraviolet information indicates the presence of such interferences.

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[3] Recommended in a private communication from J. L. Bernard, The Perkin-Elmer Corp., Silver Spring, Md.

The following recommended procedure should be useful in rapid acceptance testing of paints and in manufacturing processes to control the composition of batches or lots of alkyds or oil-alkyd vehicles. In manufacturing processes, in particular, the presence or absence of other absorbing materials would be known. It is reasonable to assume that quantitative measurements of terephthalic acid content, could be made in a manner similar to the procedures described for phthalic anhydride and isophthalic acid content.

Recommended Test Procedure

The test procedure given here is intended as a quantitative ultraviolet procedure for determining the phthalic anhydride or isophthalic acid content of alkyd resins without making any chemical separations. Absence of other high absorbance materials, at the wavelengths on which calculations are based, must be verified to use the method. The method is described for the phthalic anhydride determination; the procedure for isophthalic acid is the same except that absorbance at 282 millimicrons is used as the basis for evaluation.

Calibration

- Obtain 6 or more samples of paint or alkyd resin vehicles of the orthophthalic type in which the phthalic anhydride contents are known and cover the concentration range of interest. For paints, remove the pigments by centrifuging.
- Prepare infrared spectra of the alkyds. These may be obtained by diluting a small amount of the alkyd with chloroform, adding a few drops of the solution to the surface of a halide window, evaporating at 60° C. in a vacuum oven, and scanning with an infrared spectrophotometer. If unusual bands are evident in one sample do not attempt to use the subsequent ultraviolet procedure for that sample. Verify from the infrared spectra that these materials are typical orthophthalic alkyds.
- If the presence of tung oil is suspected, use of the Wan Test is helpful for identification. Do not use this ultraviolet procedure when tung oil, extraneous aromatics, or unsaturated dibasic acids are known to be present.
- Determine the nonvolatile solids content of each vehicle by heating it at 105° C. for 3 hours.
- Weigh accurately, by difference from a dropping bottle, an amount of vehicle sample equivalent to about 0.4 g of nonvolatile alkyd resin, and transfer to a 100-ml. round bottom flask with a standard taper neck that fits a rotating vacuum evaporator.
- Attach the flask to the rotating vacuum evaporator, evacuate, and start the rotation of the flask. Move a water bath at a temperature of 56–60°C. under the flask. Lower the flask until it is about one-half immersed in the water but does not touch the bottom of the bath. Allow evaporation to continue for 20 minutes, keeping the temperature of the water bath between 56° and 60° C.
- Turn off the vacuum connection and admit air to the assembly by removing the pressure tubing from the vacuum line. Reopen the vacuum line and turn the auxiliary ground-glass stopcock so that it is open to the air. Using a 5-ml. beaker, transfer about 45 ml. of chloroform (reagent grade) to the rotating flask. The transfer can be accomplished by repeatedly holding the 5-ml. beaker, filled with chloroform, under the glass inlet tube with one hand, and pushing the end of the pressure tubing against the vacuum-line stopcock opening with the other hand. This causes the chloroform in the beaker to be sucked through the center glass tube and into the evaporator. Shut off the vacuum line, leaving the pressure tubing disconnected. Continue rotating the flask in the water bath for 5 minutes more to insure complete solution of the residue in the flask.
- At the conclusion of the 5-minute period, raise the evaporator, dry the flask, and remove it and its contents. Transfer the contents of the flask quantitatively to a 100-ml. volumetric flask, rinse well with chloroform (under a hood), and collect the rinsings. After the flask has cooled to room temperature, dilute to the mark and mix thoroughly.
- From this stock solution, an appropriate concentration can be made by aliquoting into a 25-ml. volumetric flask and diluting this to the mark. A final concentration of 0.100 or 0.200 g/l will generally give useful absorbance values. To prepare 0.100 g/l, use:

$$\frac{0.100\times 25}{10\times \begin{array}{c}\text{weight of}\\ \text{sample}\end{array}\times\begin{array}{c}\text{nonvolatile}\\ \text{fraction}\end{array}}=\begin{array}{c}\text{milliliters of}\\ \text{stock solution}\end{array}$$

- Obtain an ultraviolet absorbance curve with a recording spectrophotometer. Before each run, establish a baseline showing the relative absorbances of the two cells containing only chloroform.
- Record the value of the corrected peak absorbance at 276 mllimicrons (nanometers). The exact position of the peak may vary from instrument to instrument.
- Repeat this absorbance measurement for each one of the range of samples used.
- On suitable graph paper, plot the points obtained from each sample with absorbance as the abscissa and percent phthalic anhydride as the ordinate. Draw a smooth curve through the points obtained. This is the calibration curve which may be used indefinitely for subsequent test samples unless there is a significant change in instrumentation.

Test sample

- Repeat the above procedure for a test sample in which the phthalic anhydride content is unknown.
- Examine the appearance of the ultraviolet spectrum of the unknown sample. If the spectrum exhibits extra peaks or shoulders not shown in the spectra from which the calibration curve was obtained, it may be well to assume that the method described here may not give acceptable results.
- From the absorbance obtained at 276 millimicrons, read the percent phthalic anhydride of the unknown from the calibration curve.

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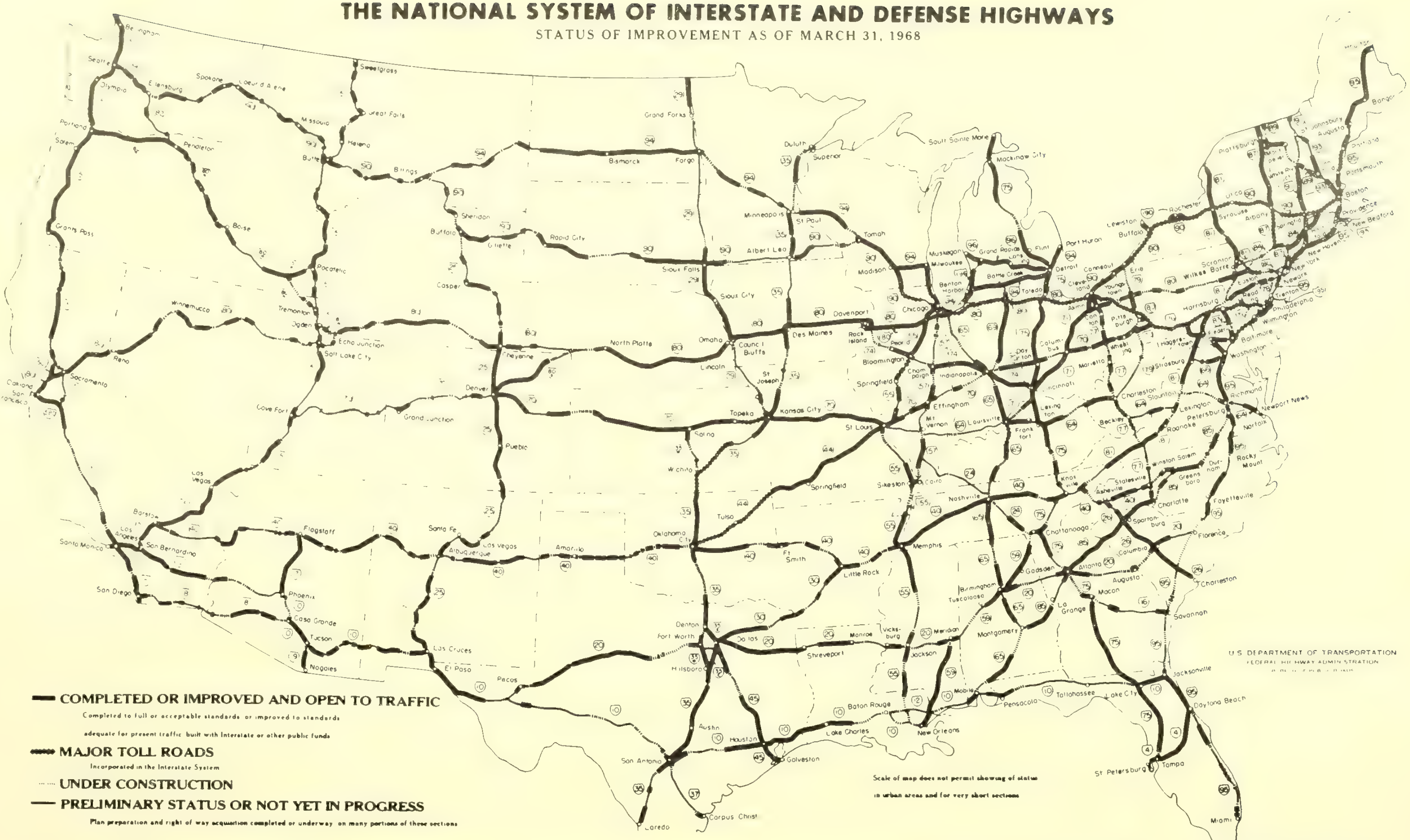

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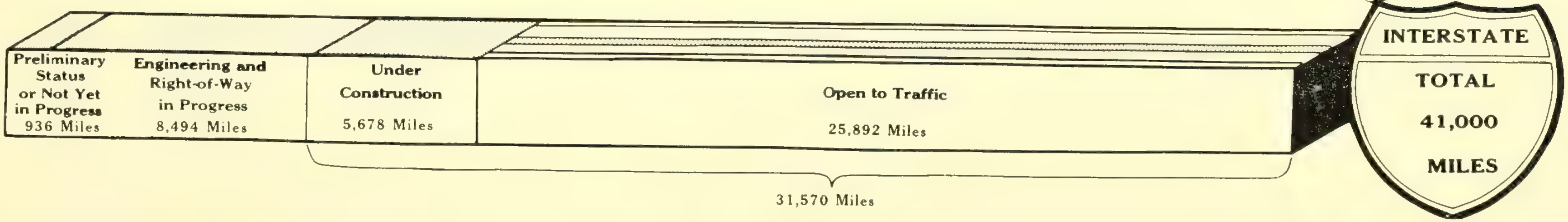

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COVER

Traffic backed up on Maryland highway U.S. 301 before it became a dual highway. Public Roads research is endeavoring to solve the problem of passing vehicles on 2-lane highways. See article beginning on opposite page.

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When deciding whether to pass or not to pass on 2-lane highways, motorists in the future may be assisted by electronic systems.

Passing Aid System I
Initial Experiments

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by DUKE NIEBUR, Highway Research Engineer,
Economics and Requirements Division

Development of a traffic system to aid motorists in passing vehicles on 2-lane rural highways is one of the chief objectives of the Public Roads research and development program. Anyone who has driven on winding, hilly, rural roads has frequently been confronted with the problem of passing a slower vehicle ahead and has either driven many laborious miles waiting for an opportune time to pass or has ventured doubtfully into the passing maneuver on the chance that it could be accomplished without mishap. If the motorist had sufficient information about conditions on the highway ahead—whether there is an oncoming vehicle in the opposite lane and whether there is enough room on the highway to pass the car ahead and clear the oncoming vehicle—the passing maneuver not only could be executed more safely, but the volume of the traffic served by the roadway would be increased by minimizing inherent delays caused by slower vehicles.

The Public Roads research and development program has turned to electronics in the search for a method of providing information that the driver needs to pass vehicles safely on 2-lane highways. Results of experiments conducted on a 2-lane roadway with an elementary passing aid system, PAS I, are described in this article. The purpose of these experiments was to determine whether drivers would rely on information supplied electronically to indicate the absence of opposing vehicles when visual sight distance was limited. Encouraging results of the experiments, as shown by acceptance of electronically indicated passing opportunities, have prompted the planning of more advanced experiments and the development of a more sophisticated passing aid system. Work is now underway on Passing Aid System II (PAS II), which is expected to be installed on 15 miles of 2-lane rural highway during 1969.

Introduction

DRIVING on high-volume, winding, 2-lane rural highways is a problem that is known to most motorists. Restricted sight distances, oncoming traffic, and adverse environmental conditions make it difficult or impossible for a motorist to pass slow vehicles, and any one of these conditions not only encourages unsafe passing attempts but also tends to decrease average vehicle speed. Furthermore, difficult passing situations, such as those existing on winding mountain roads and in dense traffic, discourage all passing attempts and encourage the unsafe practice of tailgating.

Less known to the layman is the decrease in the capacity of a highway caused by the inability of motorists to pass vehicles ahead. Even when passing sight distances are adequate, traffic volume still may reach only 30–70 percent of the roadway's capacity (volume/capacity ratio). Unfortunately, as most 2-lane rural highways do not have unrestricted passing sight distances, the volume/capacity ratio is further reduced, and

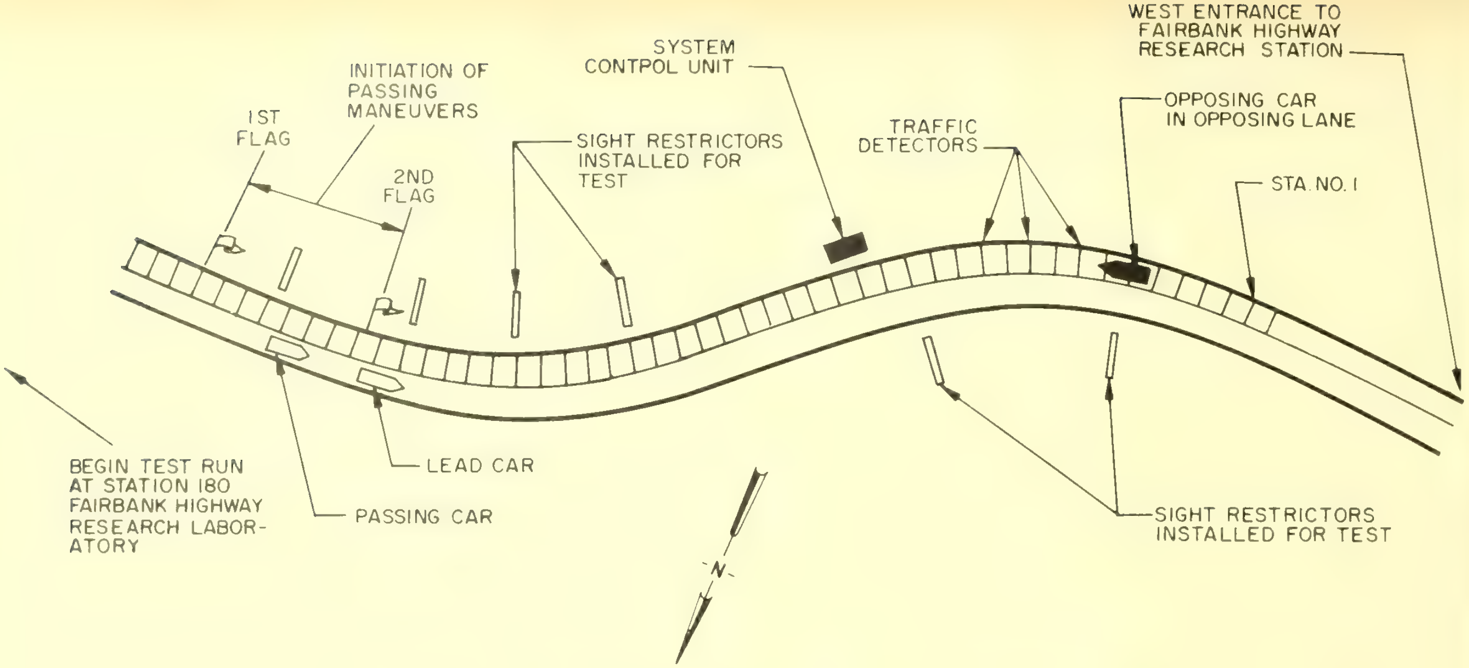

WEST ENTRANCE TO FAIRBANK HIGHWAY RESEARCH STATION
SYSTEM CONTPOL UNIT
INITIATION OF PASSING MANEUVERS
1ST FLAG
2ND FLAG
SIGHT RESTRICTORS INSTALLED FOR TEST
TRAFFIC DETECTORS
OPPOSING CAR IN OPPOSING LANE
STA. NO. 1
BEGIN TEST RUN AT STATION 180 FAIRBANK HIGHWAY RESEARCH LABORATORY
LEAD CAR
PASSING CAR
SIGHT RESTRICTORS INSTALLED FOR TEST
N

Figure 1.—Test route and operations for Passing Aid System, I.

the effect is to reduce the number of passing opportunities and create more traffic interferences—slowdowns, accidents, etc.—which reduce the service volume.

Even after the Interstate System has been completed, more vehicle-miles will be traveled on rural highways than on rural sections of the Interstate System. From this fact alone, it is evident that rural highways must be made safer. More than one-third of all accidents on these highways at present are rear-end collisions. Head-on collisions do not occur as frequently as rear-end collisions—about one-fifth of the accidents are head-on collisions—but they are likely to be more severe. Both types of accidents, however, involve the interaction of two or more drivers and their vehicles.

According to past research, a driver cannot estimate, with any degree of precision, the absolute speed of a vehicle ahead or the rate at which his own vehicle is approaching it until the two vehicles are only a few hundred feet apart. Also, according to past research, when the two vehicles are this close to each other, there is not enough time for the driver to modify his speed, especially if he is traveling at a speed typical of those on the highways today.

To avoid rear-end collisions, drivers need to be given reliable information about the speed of the vehicle ahead, or about relative speed or closure rate. Speed patterns of pairs of vehicles involved in rear-end collisions support the fact that the driver of the colliding vehicle lacked information on the vehicle ahead—more than one-third of the passenger cars were traveling at speed differences greater than 30 m.p.h. prior to collision. In normal traffic, however, less than 1 percent of pairs of cars travel at speed differences exceeding 30 m.p.h. Head-on collisions, although occurring less frequently than rear-end collisions, must be given equal attention because of their severity.

Research has shown that the average driver requires approximately 9 seconds to initiate and complete a passing maneuver on a 2-lane rural highway. Thus, if one vehicle traveling at 70 miles per hour overtakes another, a 9-second passing maneuver requires that the highway ahead be clear for a distance of more than 1,800 feet. At this distance, not only are drivers unable to estimate the relative speed of a vehicle in the opposite lane but they are incapable of determining whether that vehicle is stopped, in fact, whether its motion is toward them or away from them. Many 2-lane, bidirectional rural highway sections are without sight distances of 1,800 feet and, accordingly, are marked to prohibit passing. Moreover, the degree of precision in executing the passing maneuver has become increasingly important as traffic volumes have increased and vehicles in the opposite lane are being encountered more frequently.

Public Roads, through its national program of highway research, is endeavoring to increase travel safety on rural highways by developing methods to give the driver adequate environment information on 2-lane roadways. This information may relate to speed, acceleration, closure rate, or other information about both the vehicle ahead and the vehicle in the opposite lane.

The objective of this research is to develop a system to aid drivers solve discrimination, judgment, information, and vehicle control problems on 2-lane rural highways, and, consequently, raise highway service volumes and increase traffic safety.

Applications of electronics technology are being explored as a means to aid drivers in making judgments during overtaking and passing maneuvers. A specific application is the development of an electronic aid system that will provide the driver with information as to the presence, location, and speed of vehicles in the opposing lane. It was postulated that over a specified distance of sufficient length, considering all combinations of vehicle velocities, road grades, etc., a go or no-go type of system could be employed.

A full-scale mockup of an electronic passing aid system has been constructed and tested on the 2-lane access road to the Public Roads Fairbank Highway Research Station in McLean, Va. Summarized in this article are the concepts, experimental tests, and procedures used to determine whether drivers can and will use this electronic aid system known as Passing Aid System I, or PAS I. The willingness of drivers to use PAS I, and their ability to apply it successfully as an aid in passing vehicles on 2-lane highways, provides an indication of the advisability of developing a more advanced passing aid system.

Considerations in Developing a Passing Aid System

To speed development of passing aid systems, tentative decisions were made: (1) Drivers would be given distance information and possibly speed information or time-to-meeting information, and (2) the system had to be compatible with existing operations so that drivers of vehicles unequipped with electronic hardware could continue to use the highway.

The following basic questions need to be answered before a full-scale passing aid system can be made available for public use:

- Will drivers pass if they have information about the absence of opposing vehicles within a critical distance?

Table 1.—Test subjects used as vehicle drivers

Phase	Test subject	Age	Driving experience	Occupation	Distance driven during last 12 months: 2-lane rural highways	Distance driven during last 12 months: Freeways	Distance driven during last 12 months: City street
	Number	*Years*	*Years*		*Miles*	*Miles*	*Miles*
Preliminary Tests	1	20	3	Student	75	20	5,500
	2	20	3	Student	100	1,500	5,500
	3	21	5	Student	500	2,500	2,500
	4	22	5	Student	4,000	2,000	2,500
	5	23	7	BPR	500	2,000	2,000
	6	27	10	BPR Engineer	4,000	7,000	5,000
	7	22	6	Student	100	800	2,000
	8	30+	30	Secretary	8,000	3,000	1,000
	9	24	8	Secretary	3,000	6,000	3,000
	10	62	15	Secretary	500	500	1,000
	11	23	7	BPR	500	2,000	2,000
Experiment 1	12	29	13	BPR Engineer	14,000	6,000	10,000
	13	22	5	Secretary	5,000	10,000	5,000
	14	21	1	Student	2,500	5,000	2,500
	15	20	3	Student	1,000	2,000	2,000
	16	58	40	BPR Foreigner	800	1,700	3,000
	17			BPR Engineer			
Experiment 2	18	35	19	BPR Engineer	4,000	4,000	4,000
	19	31	10	BPR Engineer	3,000	1,000	6,000
	20	45	18	BPR Engineer	4,000	8,000	1,000
	21	26	11	BPR Engineer	8,000	8,000	2,000
	22	18	2	Student		3,000	1,000
	23	51	30	Spiritualist	3,500	7,000	3,500
	24	35	13	Housewife	4,000	8,000	3,000
	25	20	5	Student	5,000	1,000	6,000

- What criteria should be employed in determining the critical distance at which an opposing vehicle is brought to the attention of the passing driver?
- Will drivers employ other distance information about opposing vehicles in addition to the *critical distance?*
- Will drivers use opposing vehicle speed information in making a passing maneuver?
- Will drivers use time-to-meeting information in making a passing maneuver?
- How long does it take drivers to adapt to the new system?
- What instructions should be given to drivers to make it easy for them to learn how to use the new system?
- What criteria should be employed in determining how far apart the vehicle detectors shall be?
- Are there any side effects and reliability considerations that may affect the operation of the new system—driveways, cross roads, opposing cars passing other opposing cars, steep gradients, stopped vehicles, etc.?
- How will environmental conditions—rain, snow, ice, darkness—affect system operations and how can they be overcome?
- What will be the costs and the benefits of a passing aid system?
- Should information be given to drivers in visual, auditory, or tactile form?

Preliminary answers to some of these ques-ions were obtained from experimental work ith the PAS I system reported here. However, most of the questions will be answered luring PAS II operations.

Objectives of PAS I Study

The initial objectives of the first experinents with PAS I were as follows:

- To determine whether drivers, even though their sight distance is restricted, will pass when they are informed that there are no opposing vehicles within a specific distance.
- To ascertain that drivers will use speed information about the opposing vehicle to aid them in passing.
- To obtain an indication of how long it takes drivers to adapt to a new system with one set of instructions.
- To determine whether clearance distances between passing and opposing vehicles at the end of passing maneuvers are adequate, based on use of 1,300-foot signal distances.

The results of experiment 1 indicated that drivers will make selective use of passing aid information given to them by electronic means. Additional planning of more sophisticated passing aid systems is now well underway.

Description of PAS I

The first experiment was based on the use of a mockup of the Passing Aid System. The mockup, called PAS I, was installed on the access road to the Fairbank Highway Research Station and covered a distance of approximately 0.7 mile. A simplified sketch of the PAS I test setup is shown in figure 1.

The west bound direction road was used for the passing maneuver in which one vehicle, the passing car, was to overtake and pass another vehicle, the lead car, according to coded messages issued by the electronic passing aid system. The eastbound lane was used as the opposing lane in which an oncoming vehicle, the opposing car, approached the two westbound vehicles in the east lane to provide a situation that required the driver of the passing vehicle to execute the passing maneuver in time to avoid a collision or to stay in his lane behind the lead car. Sight restrictors, installed along the roadway, obstructed the driver's view of the road ahead and simulated the *blind* condition on 2-lane rural highways caused by hills and curves. Traffic detectors were spaced 44 feet apart in the lane used by opposing vehicles, and as the opposing vehicle moved over each detector, an intermittent audible signal was transmitted. The intermittent signal, which could be received by the passing car, was detectable at any point within 1,300 feet ahead of the opposing vehicle.

Four conditions could exist for the driver of the passing car: (1) No signal—the system was not operating, (2) a steady uninterrupted signal—the opposing lane was clear of traffic for at least 1,300 feet, (3) the beginning of an intermittent signal—there was a moving vehicle 1,300 feet ahead in the opposing lane, and (4) repetition of the intermittent signal—a moving vehicle was within 1,300 feet ahead in the opposing lane. The frequency/second of the intermittent signals increased with the speed of the opposing vehicle. After the beginning of the signal, the number of intermittent signals and the speed of the opposing vehicle indicated the clearance distance between the two vehicles.

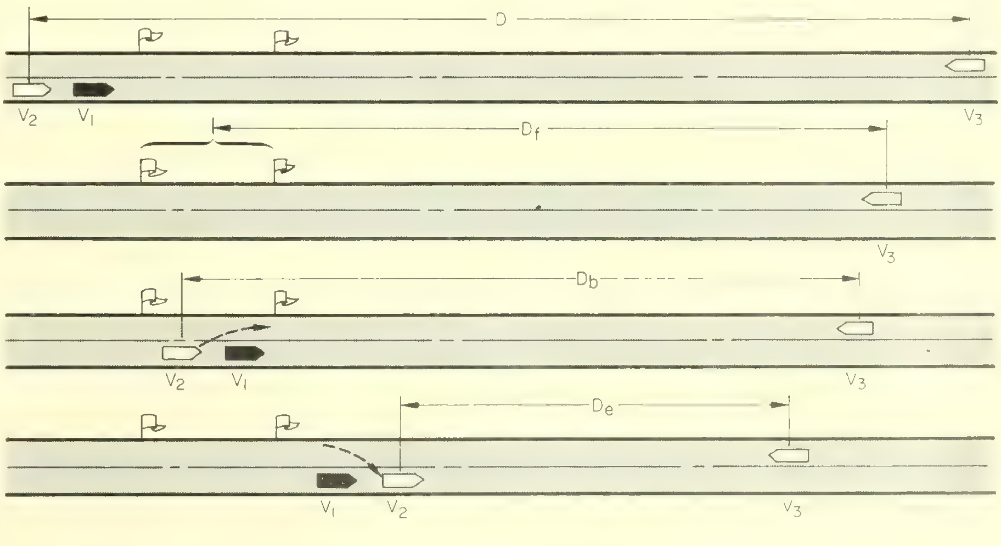


Figure 2.—Diagrammatic representation of variables.

Test Subjects and Vehicles

Test subjects used in the two experiments were obtained from the student body of George Washington University, the Bureau of Public Roads staff and the general public. Information about the drivers is shown in table 1.

The passing and opposing vehicles driven by the test subjects were 1967 4-door sedans—Dodge, Valiant, and Plymouth—with the following specifications: automatic transmission, power steering, power brakes, 6 cylinder, 225-cu. in. cylinder displacement, and 145-brake horsepower.

Table 2.—Minimum passing-sight-distance for design of 2-lane highways [1]

Design speed	Assumed passing speed	Minimum passing-sight-distance
m.p.h.	*m.p.h.*	*feet*
30	30	800
40	40	1,300
50	48	1,700
60	55	2,000
70	60	2,300

[1] Source: Blue Book, *Geometric Design Rural Highways*, 1959, p. 211.

The lead car used in experiment 1 was a 1966 4-door Ford sedan with automatic transmission. The lead car in experiment 2 was a 1967 4-door Mercury sedan with automatic transmission, power steering, power brakes, 200-brake horsepower, 8 cylinders, and cylinder displacement of 289 cu. in. In general, the test drivers considered the power and performance of the vehicles they drove to be adequate.

The combination of the driver and the vehicle he drove for the first time presented significant variables that were significant in determining the acceptance of a passing aid system.

Description PAS I Study Variables

The variables considered in the preliminary studies are itemized in the following list, and where applicable, they are shown in figure 2:

Distance
- D = distance between passing and opposing car.
- D_s = signal range generated ahead of opposing car, 1,300 feet.
- D_p = D where passing may begin—anywhere between the two flags.
- D_b = D when passing maneuver begins.
- D_e = D where passing maneuver ends.

Speed
- V_1 = speed of lead car.
- V_2 = speed of car following lead car prior to passing maneuver.
- V_3 = speed of opposing car.

Time
- T_p = time required to pass.
- T_j = time for test car to reach juxtaposition with opposing car after having completed passing maneuver.

Time, in addition to distance and speed, was observed in the hope that it would serve as a check—distance = speed × time—and be useful for the period covered by the passing maneuver when acceleration and speeds change significantly.

Two types of test runs were used in the experiments—*radio* and *control*. In the *radio*, or PAS I, test runs, the electronic passing aid system was used by the driver of the passing car to overtake and pass the lead car. The *control* test runs were made without the use of the passing aid system and were included in the experiments to provide a basis of comparison in analyzing the effectiveness of PAS I. The 2-lane test roadway had a design speed of 70 m.p.h., a posted speed limit of 30 m.p.h., and two long, 3-degree curves with a tangent between them. The posted speed limit was not in effect for the test runs.

Discussion of Variables

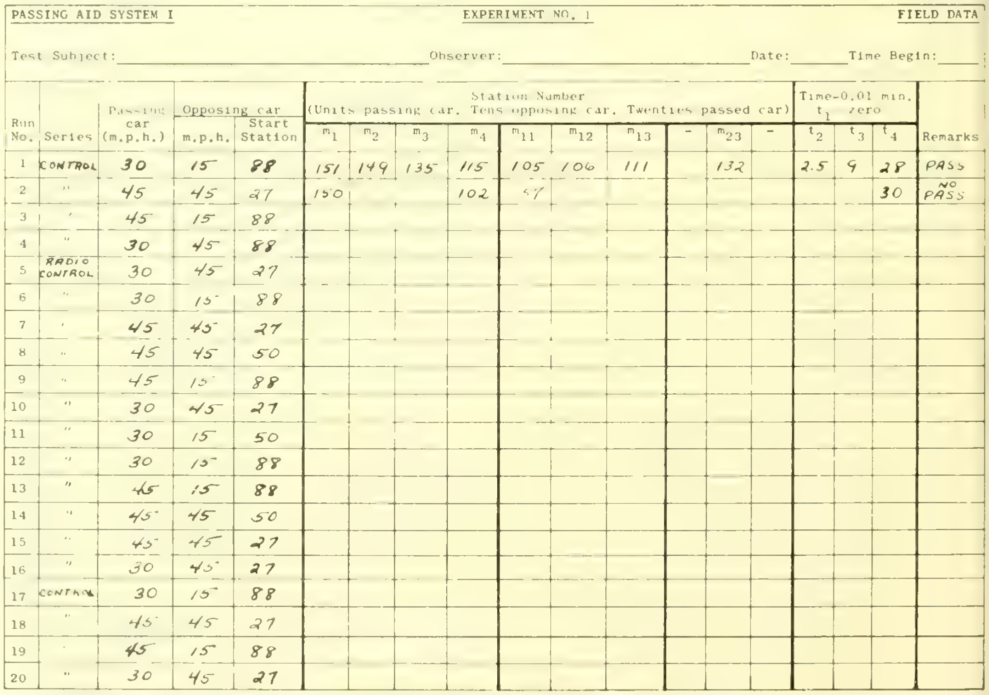
Preliminary test data, collected prior to experiments 1 and 2, indicated that several variables would have to be controlled.

The first variable was sight distance. Sight distances were so large it would have been difficult to determine whether there was any difference in the frequency of passing maneuvers between the control and simulated PAS I test runs. To decrease passing opportunity and more closely simulate driving conditions that would exist on a rural mountainous road, temporary panels were installed (see fig. 1) to restrict the sight distance. To insure comparable passing opportunities for the test and control situations, the sharpest curve, near the midpoint of the test road, was used. The part of the curve between stations 150 and 125 was selected as the section of roadway where passing maneuvers could begin. Her sight distances were 620–1,300 feet, b temporary panels reduced them to 400–5 feet. The sight distance was based on th ability of the driver in the right lane to s any part of a vehicle in the opposing lan

The second variable to be controlled w the frequency at which an opposing vehic was encountered in the passing area. For th control phase of preliminary test runs, drive were instructed to drive the way they no mally drive, but the frequency of passin maneuvers seemed abnormally high. Te drivers confirmed this by volunteering th information that normally, on the open high way, they would not pass if the sight distanc were comparable.

Three possible reasons were considered fo the incongruity between drivers' statement and actions. The first was that the test roa was always cleared of other traffic so tha passing manuevers could be based solely o the position and speed of the opposing tes vehicle. This was a definite requirement fo study of PAS I and consequently, to permi a comparison, it was also a requirement fo the control phase. Because test drivers kne there would be only one vehicle in the opposin lane and that the driver of the opposing ca would be aware of the passing maneuver, the were more willing to pass. They apparentl believed that they were not fully responsib for the passing maneuver and its possible con sequences, as they are on the open road.

Another possible reason for the discrepanc between the statements and actions of th drivers was that they were speaking in genera terms based on normal operating speeds. Fo example, table 2 gives minimum passing sight distances of 800 and 1,300 feet at speeds o



PASSING AID SYSTEM I | EXPERIMENT NO. 1 | FIELD DATA

Test Subject:________ Observer:________ Date:____ Time Begin:____

Run No.	Series	Passing car (m.p.h.)	Opposing car m.p.h.	Opposing car Start Station	Station Number (Units passing car, Tens opposing car, Twenties passed car) m_1	m_2	m_3	m_4	m_{11}	m_{12}	m_{13}	-	m_{23}	-	Time-0.01 min. t_1 zero t_2	t_3	t_4	Remarks
1	CONTROL	30	15	88	151	149	135	115	105	106	111		132		2.5	9	28	PASS
2	"	45	45	27	150			102	57								30	NO PASS
3	"	45	15	88														
4	"	30	45	88														
5	RADIO CONTROL	30	45	27														
6	"	30	15	88														
7	"	45	45	27														
8	"	45	45	50														
9	"	45	15	88														
10	"	30	45	27														
11	"	30	15	50														
12	"	30	15	88														
13	"	45	15	88														
14	"	45	45	50														
15	"	45	45	27														
16	"	30	45	27														
17	CONTROL	30	15	88														
18	"	45	45	27														
19	"	45	15	88														
20	"	30	45	27														

Figure 3.—Sample data sheet, experiment No. 1.

30 and 40 m.p.h. respectively. If these speeds were considered normal, it would have been unsafe to pass after installation of the temporary sight-restrictor panels, because the maximum sight distance available in the designated passing area was less than 550 feet. At 20 m.p.h., however, the minimum passing sight distance would have been approximately the same as the sight distance available, and the test subjects should have been willing to pass with or without the use of PAS I. The preferred approach was to study conditions in which passing maneuvers normally were not feasible, and it was decided to eliminate test runs based on a lead car speed of 20 m.p.h.

A third possible reason for the incongruity between drivers' statements and actions was that the opposing vehicle and the passing vehicle seldom were near the passing area simultaneously, and drivers may have realized that it was usually safe to pass the lead car.

Any of these possibilities or combinations of them, could have accounted for the high passing frequency in the control phase of the preliminary tests. To eliminate the first possibility, decreased driver responsibility, the following driving instructions were issued to the test subjects; these instructions replaced the game aura of the experiment with one of responsibility:

Entering the car.—"Please fasten your safety belt. The purpose of this research study is to analyze how you drive so we may develop aids to other drivers."

Test runs, control phase.—"Please start the car. Drive as you normally would on this 2-lane highway. Follow the car ahead. There will be traffic coming toward you in the opposing lane. If in your judgment you would normally pass the car ahead, you are free to do so by beginning your passing maneuver somewhere between the two red flags along the left side of the road. If you do not consider it safe to pass, continue to follow the car ahead. Drive safely. Take no chances. Drive in a manner similar to the way you drive on the highway. Any questions?"

Test runs, passing aid phase.—"When you hear a continuous tone, from your radio receiver, the opposing lane is clear of moving traffic for at least ¼ mile. When a vehicle is moving toward you in the opposing lane closer than ¼ mile, you will hear beeps on the radio. If you desire to pass, you may use the radio signals to aid you in deciding whether or not to pass.

"As before, if you do choose to pass the car ahead, the passing maneuver should start in the area between the red flags. Any questions?"

The second possibility was eliminated by discarding the 20-m.p.h. test runs, mentioned earlier, and the third by increasing, for each test subject, the percentage of runs in which there was no opportunity to pass in the passing area. For the *no-passing* situation to occur in the designated passing area, it was necessary to specify not only the lead car and opposing vehicle speeds, but also the stations from which the vehicles would begin each test run.

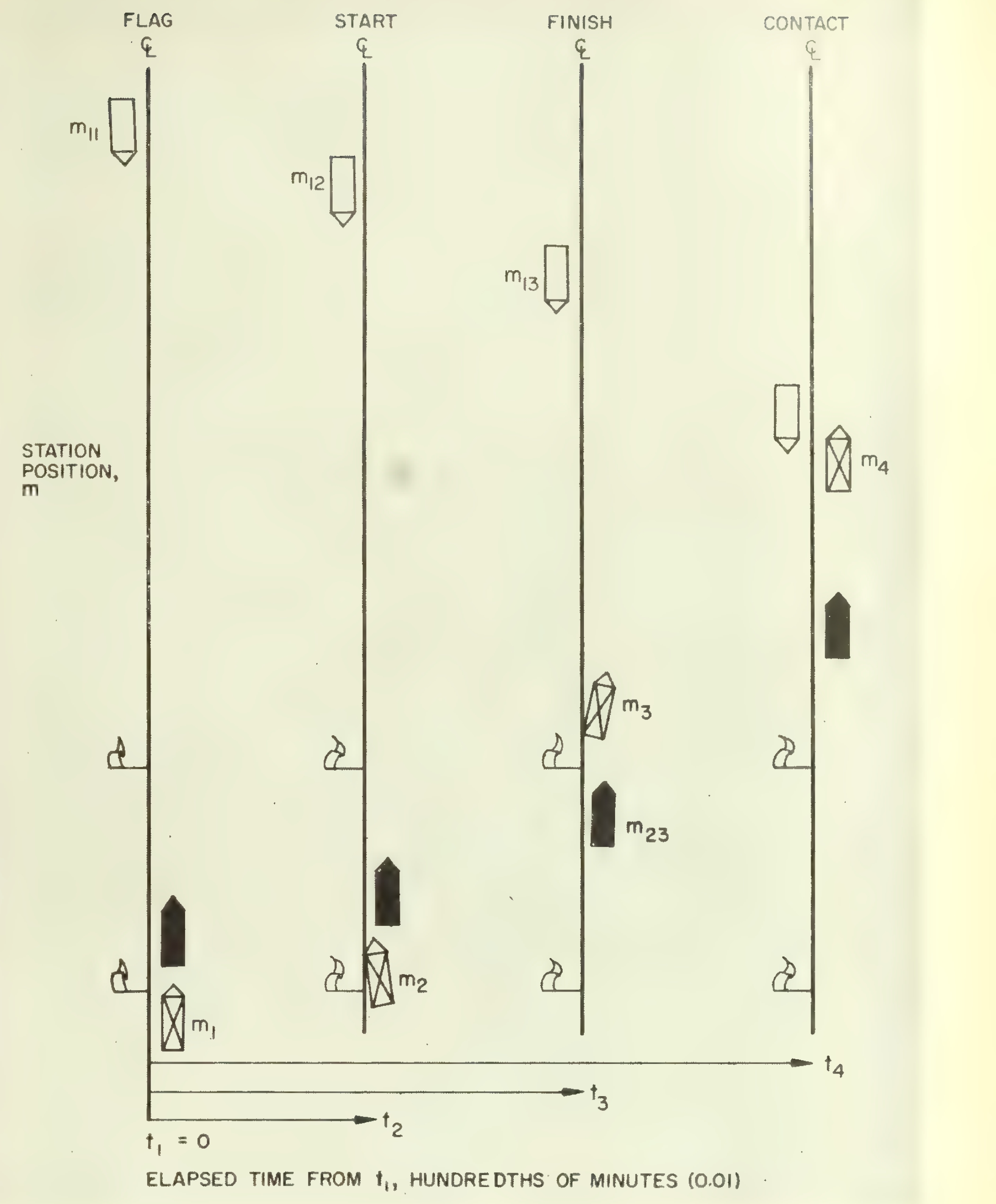


Figure 4.—Vehicle positions at which station numbers and elapsed time was recorded.

Experiments and Procedure

Data were collected in two series of tests—experiment 1 and experiment 2. In both experiments the test subjects were used in pairs. For a test run, one subject would operate the passing vehicle and the other subject the opposing car. For the next test run, the drivers exchanged assignments so that each driver was used coming and going in each pair of runs.

Experiment 1

Data for experiment 1 were recorded on the form shown in figure 3. The first five vertical colums at the left contain the previously discussed control variables. The *run number* indicates the individual trips on the test road during which a passing maneuver could occur. The column originally indicated the trip sequence for both drivers, but midway through experiment 1, this arrangement was determined to be undesirable, as one driver of each pair of drivers would operate the opposing vehicle in one run, then operate the passing vehicle in the next run under identical test conditions. Consequently, he could recall the starting position of the opposing vehicle, its speed, the clearance distance available for passing, or any one of these factors, to formulate a predetermined pass or no-pass decision. In the field it was decided to eliminate the

Table 3.—Data summary for experiment 1, test subjects Nos. 12–17 [1]

Speed combination		Begin run		Opposing lane clear for more than 1,300 ft. at 1st flag	Pass/No-pass frequency				Passing percentage			
					Control		PAS I		Control		PAS I	
Lead car	Opposing car	Lead car	Opposing car		Pass	No-pass	Pass	No-pass	Test	Total	Test	Total
m.p.h.	m.p.h.	Station No.	Station No.		Number	Number	Number	Number	Percent	Percent	Percent	Percent
30	15	180	27	Yes	2	2	1	0	50		100	
		180	50	Yes	0	0	6	2			75	
		180	88	No	4	3	3	5	57		38	
				Total passing percent						55		59
30	45	180	27	Yes	2	2	12	3	50		80	
		180	50	No	0	0	0	0				
		180	88	No	1	5	1	0	17			
				Total passing percent						30		81
45	15	180	27	Yes	0	0	0	0				
		180	50	Yes	0	0	0	0				
		180	88	No	1	7	1	6	12		14	
				Total passing percent						12		14
45	45	180	27	Yes	3	3	7	3	50		70	
		180	50	Yes	0	0	5	4			56	
		180	88	No	0	5	1	1			50	
				Total passing percent						27		61
				Total observed data	13	27	37	24		32		61

[1] Test subjects are listed in table 1.

Table 4.—Data summary for experiment 2, test subjects Nos. 18–23 [1]

Speed combination		Begin run		Opposing lane clear for more than 1,300 ft. at 1st flag	Pass/No-pass frequency				Passing percentage			
					Control		PAS I		Control		PAS I	
Lead car	Opposing car	Lead car	Opposing car		Pass	No-pass	Pass	No-pass	Test	Total	Test	Total
m.p.h.	m.p.h.	Station No.	Station No.		Number	Number	Number	Number	Percent	Percent	Percent	Percent
30	15	180	27	Yes	2	4	4	2	33		66	
		180	88	No	0	6	2	4	0		33	
				Total passing percent						17		50
30	45	180	27	Yes	1	5	5	1	17		83	
		180	88	No	2	4	1	5	33		16	
				Total passing percent						25		50
45	15	180	27	Yes	0	6	0	6	0		0	
		180	88	No	0	6	0	6	0		0	
				Total passing percent						0		0
45	45	180	27	Yes	0	6	1	5	0		16	
		180	88	No	0	6	0	6	0		0	
				Total passing percent						0		8
				Total observed data	5	43	13	35		10		27

[1] Test subjects are listed in table 1.

Table 5.—Data summary for experiments 1 and 2 combined, test subjects Nos. 12–17 and 18–23 [1]

Speed combination		Begin run		Opposing lane clear for more than 1,300 ft. at 1st flag	Pass/No-pass frequency				Passing percentage			
					Control		PAS I		Control		PAS I	
Lead car	Opposing car	Lead car	Opposing car		Pass	No-pass	Pass	No-pass	Test	Total	Test	Total
m.p.h.	m.p.h.	Station No.	Station No.		Number	Number	Number	Number	Percent	Percent	Percent	Percent
30	15	180	27	Yes	4	6	5	2	40		71	
		180	88	No	4	9	5	9	31		36	
				Total passing percent						35		48
30	45	180	27	Yes	3	7	17	4	30		81	
		180	88	No	3	9	2	5	25		29	
				Total passing percent						27		68
45	15	180	27	Yes	0	6	0	6	0		0	
		180	88	No	1	13	1	12	7		8	
				Total passing percent						5		5
45	45	180	27	Yes	3	9	8	8	25		50	
		180	88	No	0	11	1	7	0		12	
				Total passing percent						13		37
				Total observed data	18	70	39	53		26		74

[1] Tests subjects are listed in table 1.

possibility of predetermined decisions by a random selection of each succeeding test run from those remaining to be made. Accordingly, the data in experiment 1 were considered to be free of predetermined pass or no-pass decisions.

In the second column of figure 3, *control*, means without the use of the passing aid system, and *radio*, means with use of the passing aid system. The data under *pass. car* are the speeds of the lead car and passing car prior to starting the passing maneuver. The speed of the opposing car and the station from which it started are given in the next two columns for each test run. In all test runs, the lead car and the test vehicle started from a stationary position at station 180. The results of preliminary test runs, before PAS I was operational, were the basis for determining the preselected variables.

The instrumentation for the experimental procedure was simple. Distances were determined by relating vehicle positions to the electronic detector stations numbered consecutively from the west end of the test road; stations were 22 feet apart. Speeds for the lead car and the opposing car were preselected, and the drivers accelerated to the constant speeds and maintained them by referring to the speedometer in the vehicles. Two-way radios were used to communicate among the three vehicles and disseminate the following information: *road clear,* meaning the route is clear for the test run; *flag,* meaning the beginning of the length of roadway where a passing maneuver could be initiated; *start,* meaning the driver has started the passing maneuver; and *finish,* meaning the driver has completed the passing maneuver and has returned to the right lane. The relations between vehicle positions and elapsed times are shown in figure 4. The term *contact,* shown at time t_4, means that the passing vehicle and the opposing vehicle are at the same position on the roadway after completion of the passing maneuver.

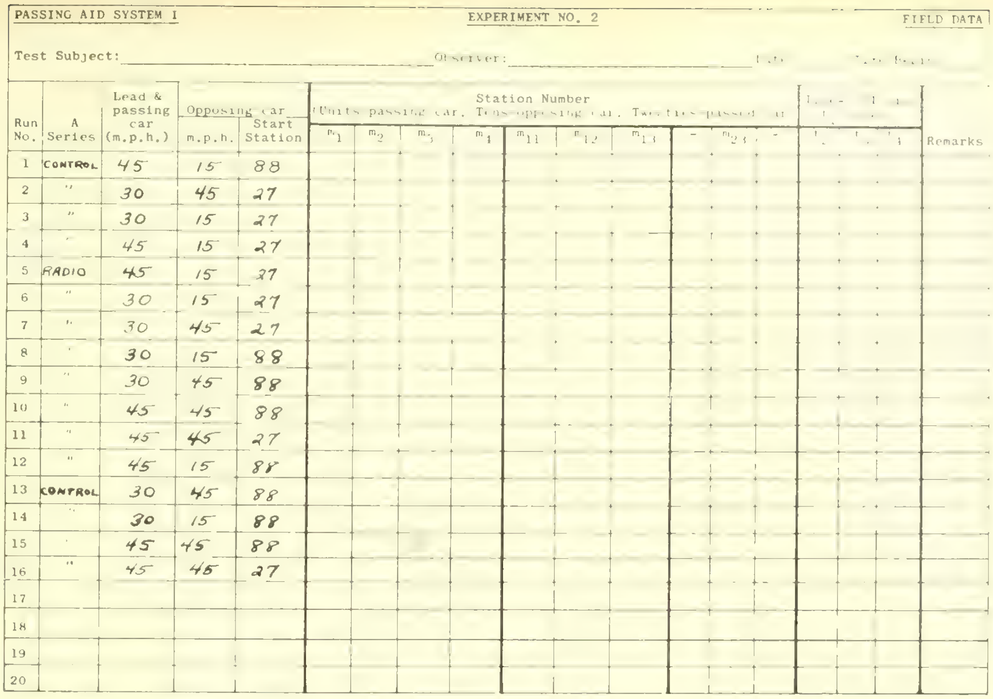
In figure 3, m_1, m_2, m_3, and m_4 are the station locations of the test vehicle at *flag, start, finish,* and *contact* respectively; m_{11}, m_{12}, and m_{13} are the station locations of the opposing car when the passing car signals *flag, start,* and *finish;* and m_{23} is the recorded position of the lead car at the finish signal. In the time columns of figure 3, t_2, t_3, and t_4 are the elapsed times in hundredths of a minute (0.01) from *flag* to *start, finish,* and *contact,* respectively.

Experiment 2

Experiment 2 was basically a continuation of experiment 1, but was different in two respects: (1) Pass or no-pass signals, that could be increased in volume and include an accompanying light signal on the dashboard of the car, were provided for the drivers, (2) The number and distribution of runs, were adjusted according to control variables so that there would be matching *control* and *radio* (PAS I) runs.

The predetermined orders of runs for experiment 2, which were based on the theory of random numbers, are shown in figures 5 and 6. These two orders, A series and B series, eliminated possible bias that could have occurred if the test runs had been selected in the field, and decreased the possibility that drivers would know the position of the opposing car. Test runs in A series and B series were similar, only the sequence was different.

The percentage of test runs that were passing maneuvers in the control and PAS I phases are shown in figures 8, 9, and 10 respectively for experiment 1, experiment 2, and experiments 1 and 2 combined. Data points on the graphs are the percentage subtotals shown on tables 3, 4, and 5. For example, in table 6 and figure 8 it is shown that for a lead car speed of 30 m.p.h., an opposing car speed of 15 m.p.h., and three opposing car starting stations, 55 percent of the control test runs were passing maneuvers. To make valid comparisons of passing percentages between the control and PAS I phases, the proportions of the runs assigned to the different opposing car starting stations would have to be equal for both

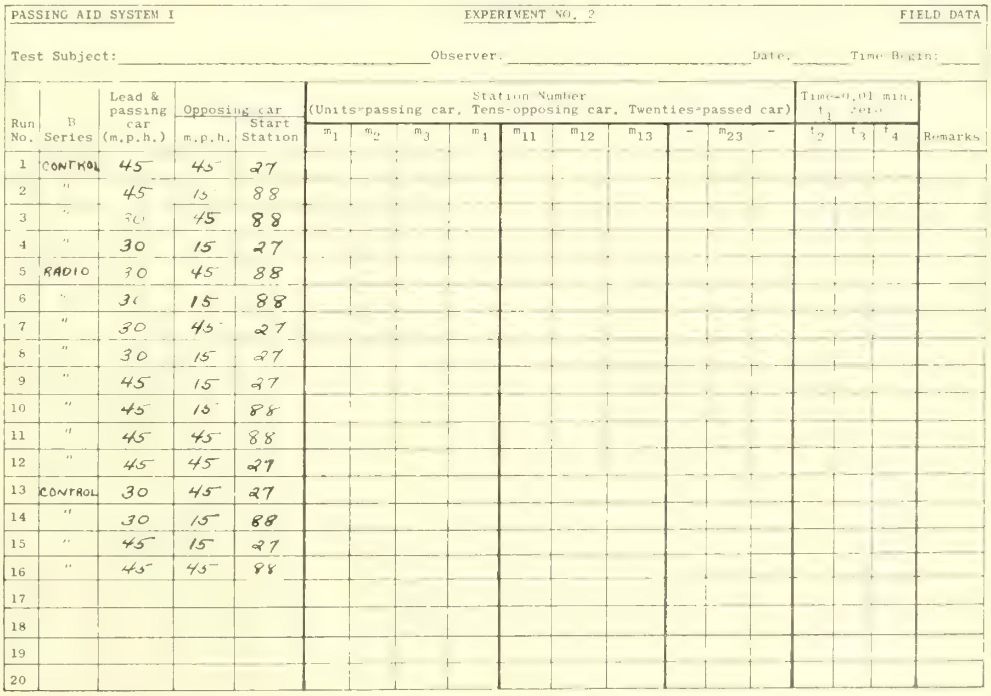


PASSING AID SYSTEM I — EXPERIMENT NO. 2 — FIELD DATA

Test Subject: ______ Observer: ______ Date ______ Time Begin: ______

Run No.	A Series	Lead & passing car (m.p.h.)	Opposing car		Station Number (Units=passing car, Tens=opposing car, Twenties=passed car)										Time=0.01 min. t_1 zero			Remarks
			m.p.h.	Start Station	m_1	m_2	m_3	m_4	m_{11}	m_{12}	m_{13}	-	m_{23}	-	t_2	t_3	t_4	
1	CONTROL	45	15	88														
2	"	30	45	27														
3	"	30	15	27														
4	"	45	15	27														
5	RADIO	45	15	27														
6	"	30	15	27														
7	"	30	45	27														
8	"	30	15	88														
9	"	30	45	88														
10	"	45	45	88														
11	"	45	45	27														
12	"	45	15	88														
13	CONTROL	30	45	88														
14	"	30	15	88														
15	"	45	45	88														
16	"	45	45	27														
17																		
18																		
19																		
20																		

Figure 5.—Sample data sheet, experiment No. 2, Series A.



PASSING AID SYSTEM I — EXPERIMENT NO. 2 — FIELD DATA

Test Subject: ______ Observer. ______ Date. ______ Time Begin: ______

Run No.	B Series	Lead & passing car (m.p.h.)	Opposing car		Station Number (Units=passing car, Tens-opposing car, Twenties=passed car)										Time=0.01 min. t_1 zero			Remarks
			m.p.h.	Start Station	m_1	m_2	m_3	m_4	m_{11}	m_{12}	m_{13}	-	m_{23}	-	t_2	t_3	t_4	
1	CONTROL	45	45	27														
2	"	45	15	88														
3	"	30	45	88														
4	"	30	15	27														
5	RADIO	30	45	88														
6	"	30	15	88														
7	"	30	45	27														
8	"	30	15	27														
9	"	45	15	27														
10	"	45	15	88														
11	"	45	45	88														
12	"	45	45	27														
13	CONTROL	30	45	27														
14	"	30	15	88														
15	"	45	15	27														
16	"	45	45	88														
17																		
18																		
19																		
20																		

Figure 6.—Sample data sheet, experiment No. 2, Series B.

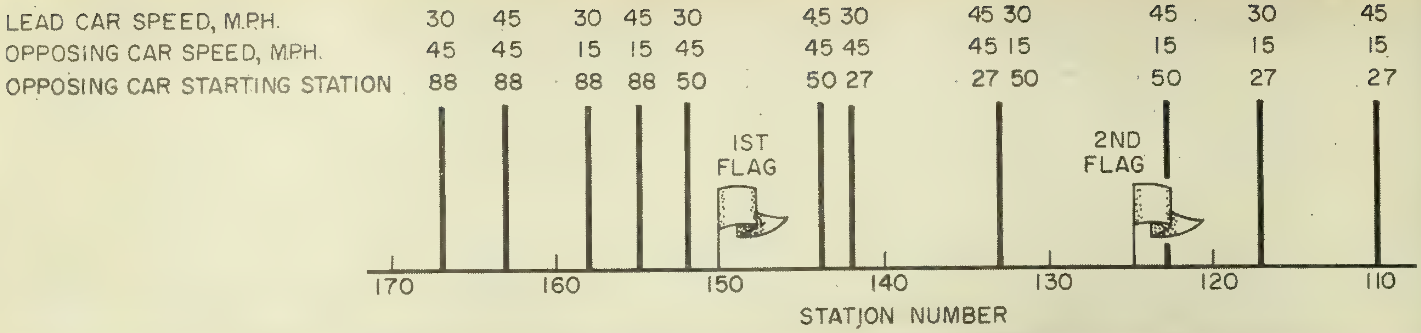


Figure 7.—Estimated position of passing car at which signal from opposing car would first be received.

phases. This requirement was met for experiment 2 (fig. 9) but was not for the other experiments, which may explain the possible misalinement of the control phase curves of figure 8. The data in all three figures indicate the apparent increased percentage of test runs having passing maneuvers for transitions from the control phase to the PAS I phase.

Statistical Tests

Chi square tests and confidence limit intervals were used to determine the statistical significance of the results. An advantage of the chi square test is the *yes* or *no* answer obtained. However, in situations where data do not meet minimum requirements, the chi square test is not applicable. Confidence limit bands can be based on any size sample, but conclusions can vary with interpretation of the bands. Both approaches were used with emphasis given to the one considered most applicable to the particular analysis being made.

Chi square tests

The chi square statistic takes into account the similarities of samples that occur by chance alone, regardless of whether the samples are from the same or different populations. A calculated value of the chi square, based on observed data, can be compared to standard tabulated values of chi square shown in textbooks on statistics (*1*).[1] Depending on the percentage level of confidence desired, the comparison can infer whether any difference in two samples is likely to have occurred by chance alone. If an existing difference did not occur by chance alone, then the difference is significant.

The chi square tests used in this report were based on the use of 2 × 2 tables (1 degree of freedom), and a tabulated value of chi square equal to 3.84 for the 95-percent confidence level. The 95-percent confidence level is commonly used and accepted in research.

Confidence intervals

To estimate the mean of a population, it is helpful to have not only a sample mean but also a measure of the margin of error of the sample mean. A way to do this is to specify a zone, based on the sample mean, within which the population mean lies. This zone is called a confidence interval, and the end points of this interval are called confidence limits. The probability that the interval will include the population mean is stated as a percentage and is referred to as the confidence level. The 95-percent confidence level was used in the research reported here.

The control phase of the experiments was the population, or real world, used as a basis of comparison. Because the control phase was also a sample, the test basically was a comparison of two sample intervals. If the ranges of the two confidence intervals were generally similar, the samples were from the same population. If the ranges of the confidence intervals were generally different, then the samples were from different populations.

Confidence limits for each proportion were obtained from an Ordnance Engineering Design Handbook (*2*). The upper and lower confidence limits were obtained from tables for samples of fewer than 30 observations and from graphs requiring interpolation for samples of more than 30 observations.

The results of the statistical tests have been assembled as *yes* or *no* answers to questions given in table 6.[2] For example, if a statistical test determined that a slight increase in passing frequency was insignificant, the answer in

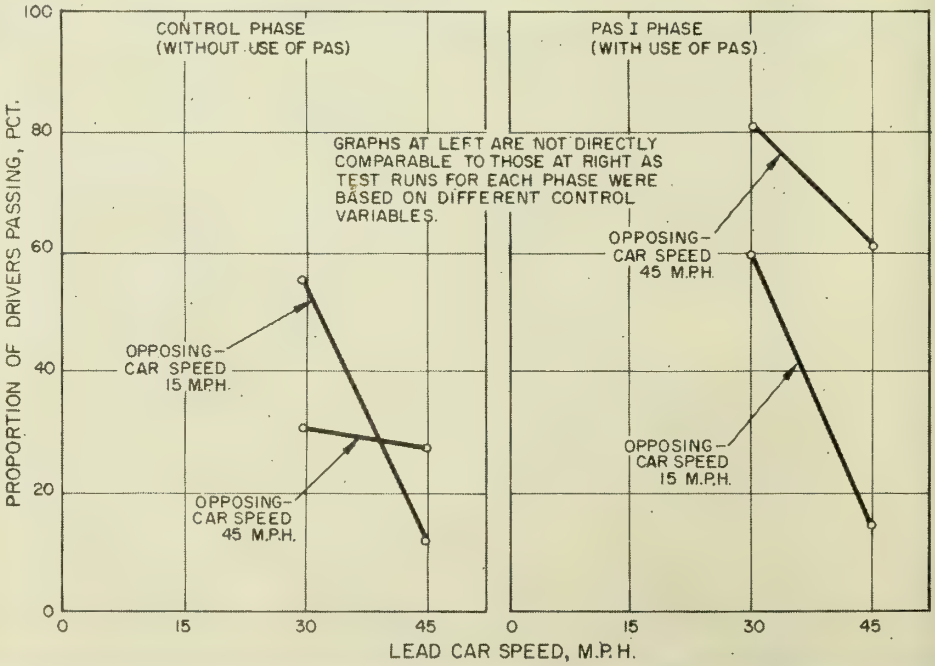


Figure 8.—Percentage of drivers passing—with and without PAS—experiment 1.

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[1] The italic numbers in parentheses identify the references listed on p. 76.

[2] Statistical tests that were made for each analysis have been assembled and are available from the Office of Research and Development, Bureau of Public Roads, ℅ Managing Editor, *Public Roads* Magazine.

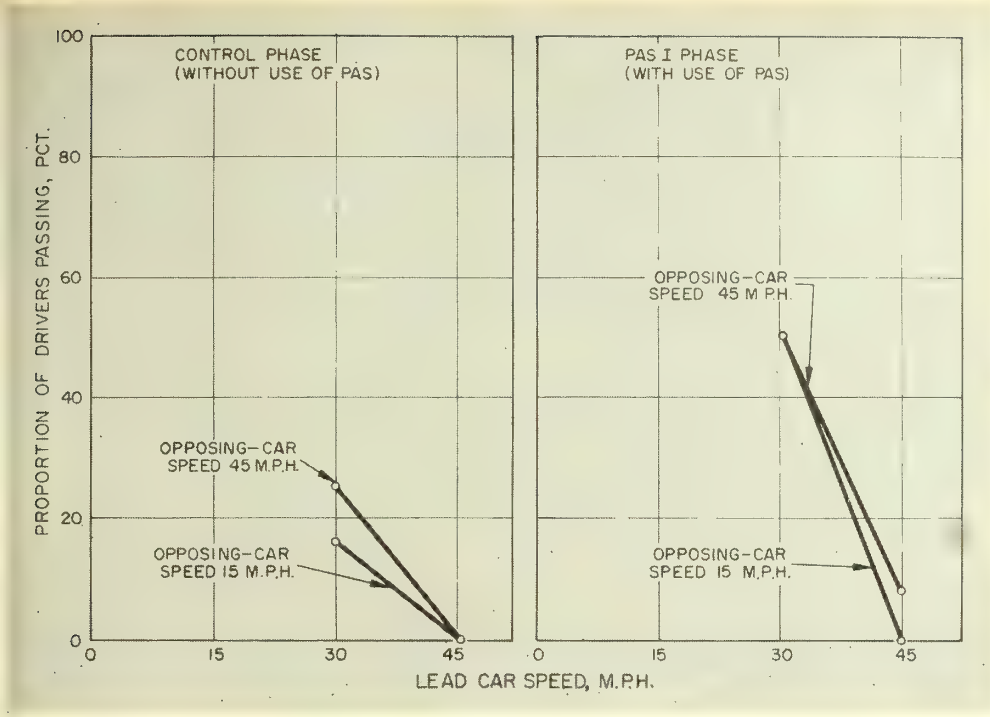


Figure 9.—Percentage of drivers passing—with and without PAS—experiment 2.

table 6 and in the following discussion would state that there was no increase in passing frequency.

Analyses of Statistical Data

A summary of the analyses of the primary data, frequency of passing maneuvers, is given in table 6. The analysis number at the left is followed by the question that the analysis poses. Answers to the question, based on use of the chi square test and confidence intervals for each experiment, are indicated in the columns at the right.

Experiment 2 was the only experiment that had proportionate distribution of test runs with regard to the control variables for each sample. Comparisons of data for experiment 2 were therefore favored over those for experiment 1 and 1 and 2 combined.

Analysis 1

The first analysis was made to determine whether PAS I, compared to the control phase, increased the percentage of passing. The comparison for each experiment was based on all data. Each statistical test applied to the different experiments indicated that use of PAS I did increase the percentage frequency of passing maneuvers.

Analysis 2

The analysis of PAS I, compared to the control phase for lead car speeds of 30 m.p.h., and all the statistical tests used, indicated that PAS I increased the passing percentage.

Analysis 3

Compared to the control phase for lead car speeds of 45 m.p.h., the analysis of PAS I failed to show conclusively that it increased passing percentage. Chi square tests were limited because of sample distribution and/or the minimum data criteria for the test. The use of confidence intervals indicated that PAS I increased passing percentage for experiment 1 and 1 and 2 combined, both of which were unbalanced samples. The one reliable test for this analysis, the use of confidence limits on experiment 2, indicated that PAS I did not increase passing percentage.

Analysis 4

The analysis, based solely on the use of PAS I indicated that when lead car speed was increased from 30 to 45 m.p.h., there was a decrease in passing frequency. The statistical tests based on experiment 1 produced answers to the contrary, or answers with doubtful conclusions because of the unbalanced sample distribution. Experiment 2, with balanced sampling, produced the most reliable conclusions which were supported by the conclusions from the combined data of experiments 1 and 2. The accepted conclusion is reasonable, considering the fact that as traffic speed increased, fewer passing maneuvers were required to maintain desired speed.

Analysis 5

When signaled clearance distance at the first passing opportunity was more than 1,300 feet, the analysis of PAS I indicated an increase in passing frequency with the use of PAS I, when compared to the control phase. The statistical tests were in agreement for each of the experiments.

Analysis 6

When signaled clearance distance was less than 1,300 feet, the analysis of PAS I compared to the control phase indicated no change in passing frequency. Statistical tests were in agreement for all experiments, though it should be noted that data were below the minimum required for chi square tests in experiments 1 and 2.

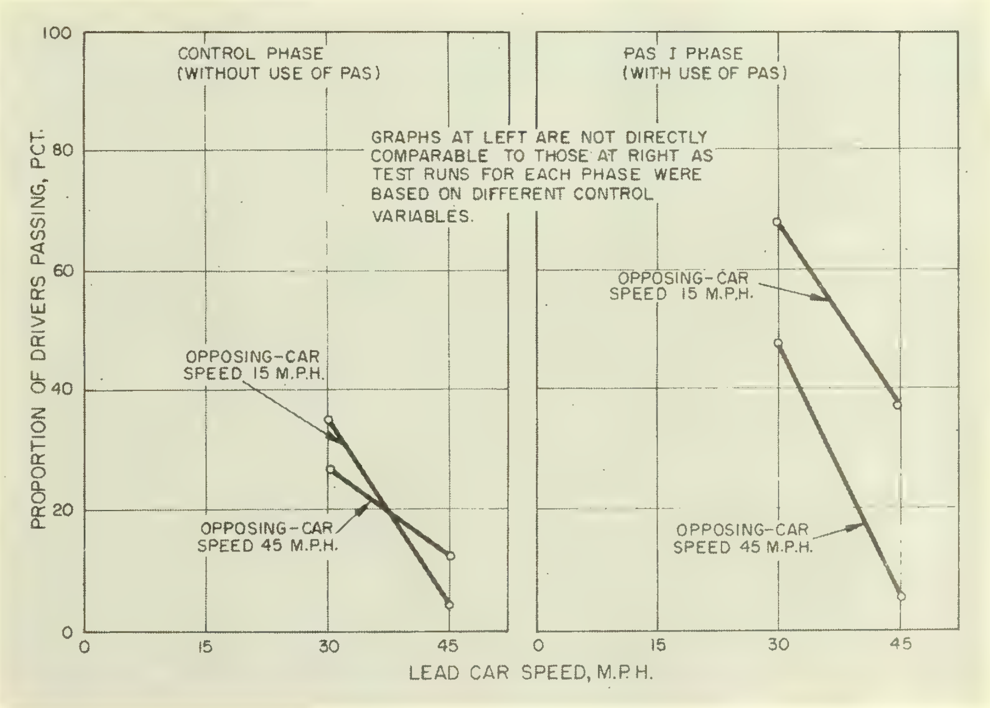


Figure 10.—Percentage of drivers passing—with and without PAS—experiments 2 and 3 combined.

Table 6.—Results of statistical tests

Analysis No., description, and question	Answers for each experiment					
	Chi square test			Confidence interval		
	1 [1]	2	1 and 2 [1] combined	1 [1]	2	1 and 2 [1] combined
1 Did PAS I, compared to control phase, show larger percentage of passing?	yes	yes	yes	yes	yes	yes
2 Did PAS I, compared to control phase for lead car speed of 30 m.p.h., show larger passing percentage?	yes	yes	yes	yes	yes	yes
3 Did PAS I, compared to control phase for lead car speed of 45 m.p.h., show larger passing percentage?	yes	(2)	yes	yes	no	yes
4 Did passing percentages decrease with use of PAS I and increase in lead car speed (30–45 m.p.h.)?	no	yes	yes	?	yes	yes
5 Did PAS I, compared to control phase, increase passing percentage when signal clearance exceeded 1,300 ft.?	yes [2]	yes	yes	yes	yes	yes
6 Did PAS I, compared to control phase, give similar passing percentage when signal clearance was less than 1,300 ft.? [3]	yes [2]	(2)	yes	yes	yes	yes
7 Did PAS I, compared to control phase show increase in passing for:						
a Lead car speed of 30 m.p.h., opposing car speed of 15 m.p.h., opposing car starting station 27						yes
b Lead car speed of 30 m.p.h., opposing car speed of 15 m.p.h., opposing car starting station 88						no
c Lead car speed of 30 m.p.h., opposing car speed of 45 m.p.h., opposing car starting station 27						yes
d Lead car speed of 30 m.p.h., opposing car speed of 45 m.p.h., opposing car starting station 88 [3]						no
e Lead car speed of 45 m.p.h., opposing car speed of 15 m.p.h., opposing car starting station 27						no
f Lead car speed of 45 m.p.h., opposing car speed of 15 m.p.h., opposing car starting station 88 [3]						no
g Lead car speed of 45 m.p.h., opposing car speed of 45 m.p.h., opposing car starting station 27						no
h Lead car speed of 45 m.p.h., opposing car speed of 45 m.p.h., opposing car starting station 88 [3]						no

[1] Conclusions weakened by sample distributions.

[2] Does not satisfy criteria for chi square test.

[3] Signaled clearance distance is less than 1,300 ft. when the passing vehicle reaches the permitted passing area.

Table 7.—Clearance distance between passing and opposing vehicles at completion of passing maneuver, experiments 1 and 2 combined

Control variables			Phase	Test runs			Average clearance distance
Speed of lead car	Speed of opposing car	Beginning station for opposing car		Total	With passes	With passes and data	
m.p.h.	*m.p.h.*	*number*		*number*	*number*	*number*	*feet*
30	15	27	Control	12	6	6	1,364
			PAS I	8	6	6	1,408
30	15	88	Control	14	5	3	180
			PAS I	14	5	5	374
30	45	27	Control	11	4	3	300
			PAS I	22	18	17	814
30	45	88	Control	13	3	2	0
			PAS I	8	2	1	242
45	15	27	Control	8	0	0	....
			PAS I	7	0	0	
45	15	88	Control	14	1	1	154
			PAS I	13	1	1	132
45	45	27	Control	13	4	3	968
			PAS I	16	8	7	726
45	45	88	Control	10	0	0	
			PAS I	8	1	0	

Analysis 7

In this analysis the confidence intervals were developed for the control and PAS I phases of eight combinations of lead car speed, opposing car speed, and opposing car starting station. The question for each combination was "does the use of PAS I increase the frequency of passing maneuvers when compared to the control phase?" Results of statistical tests can be categorized as follows:

(1) Those combinations based on the opposing car starting from station 88 showed no increase in passing maneuvers. The signaled clearance distance at the first passing opportunity for each combination was less than 1,300 feet. The findings were in agreement with those of analysis 6.

(2) Those combinations, based on the opposing car starting from station 27, showed an increase in passing maneuvers when the lead car speed was 30 m.p.h. and no increase in passing maneuvers when the lead car speed was 45 m.p.h. The findings were in agreement with those findings of analyses 2 and 3, respectively.

Clearance at End of Passing Maneuvers

One of the objectives of the experiment was to determine whether clearance distanc between the passing and opposing vehicl at the end of the passing maneuver, base on use of the 1,300-foot clearance distance was adequate. Average clearance distances according to data from combined experiment 1 and 2, are shown in table 7. Data in th table are classified at the left by the contro variables: lead car speed, opposing car speed and opposing car starting station. For bot the control and PAS I phases each classification shows the total number of test run observed, number of test runs that wer passing maneuvers, number of test runs wit passes for which clearance data were obtained and average clearance distance between th passing and opposing vehicle at the end o the completed passing manuever.

The use of PAS I when compared to th control phase, based on the lead car spee of 30 m.p.h., increased the clearance distanc at the end of the passing maneuvers. Fo lead car speeds of 45 m.p.h., the clearanc distance decreased.

Summary

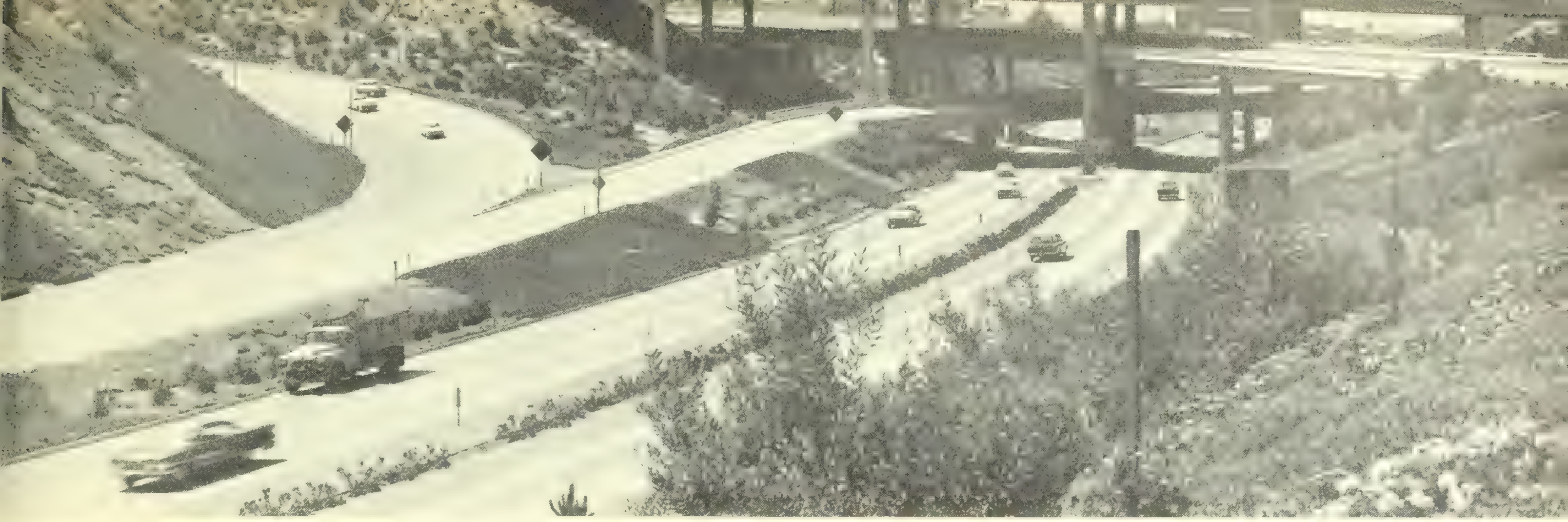
A full-scale mockup of a Passing Ai System (PAS I) was installed and tested on a short section of 2-lane highway. Results of a limited experiment showed that when sight was restricted to a distance considerably below the 800 feet of passing sight distance required for a design speed of 30 m.p.h., drivers made selective use of passing-distance information given to them electronically. At operating speeds of 30 m.p.h., drivers made significant use of PAS I when clear distances exceeding 1,300 feet were indicated electronically, and the passing percentage was substantially increased (see fig. 9). At 45 m.p.h., drivers used PAS I less frequently The sight distance for passing at 45 m.p.h. is approximately 1,500 feet, but the present design of PAS I provided only 1,300 feet for passing.

The one set of instructions used in thes experiments introduced drivers to the passing aid system but did not adapt them to it. Use of the passing aid system is required. The experience gained was too limited to conclude that the drivers had satisfactorily adapted to the system.

Based on use of PAS I, clearance distances between the passing vehicle and opposing vehicle at the end of the passing maneuver were adequate when lead car speeds were 30 m.p.h., but inadequate when lead car speeds were 45 m.p.h. The control phase was used as the basis for this conclusion.

Results of the study were favorable for the development and use of passing aid systems. Although the range of conditions under which such systems would be useful may not be as broad as anticipated, further testing with passing aid systems is justified.

(*Continued on p. 76*)



Deviation from the mean travel speed on the Interstate System increases the probability of involvement in an accident.

Interstate System Accident Research Study II, Interim Report II

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by JULIE ANNA CIRILLO,
Mathematician,
Traffic Systems Division

Introduction

THE RESULTS of an analysis of the effects that speed variance among vehicles, level of enforcement, and interchanges have on accident and involvement rates are presented in this report—the second interim report on data collected for the Interstate System Accident Research Study II. The objectives of the research and related study procedures were described in Interim Report I (*1*).[1] The data used in the analysis presented here were collected by 20 State highway departments (see fig. 1).

Speed Variance Among Vehicles

It has been shown in past research that the severity of a given accident will increase as the speed of the vehicles prior to collision increases (*2, 3*). However, the chance of being involved in an accident, at least on 2- and 4-lane main rural highways, having no control of access (*2*), has been shown to be related to speed variance, or deviation from the speed of the traffic stream. It was sought in the analysis reported here to determine whether speed variance contributes to accident involvement on the Interstate System as well. Only accidents occurring between 9 a.m. and 4 p.m. on mainline units were used in the analysis to correspond with the speed data and vehicle classification data collected for the same period. For this study, a mainline study unit is defined as any section of the Interstate highway that is not more than 10,000 feet in length and homogeneous throughout, with respect to its geometric characteristics. Speed data were not obtained during the hours from 4 p.m. to 9 a.m. Speed change lanes, although classified as separate units, were included in the category of mainline units. Ramps, crossroad units, frontage roads, and other units were not included in the analysis. To further reduce the number of variables, 2-lane two-direction mainline study units were eliminated from the analysis; however, both urban and rural sections were studied but were not separated.

In determining the effect of speed variance—not used here in the statistical sense—only rear-end and angle collisions and same-direction sideswipe accidents, occurring between 9 a.m. and 4 p.m., were considered. The assumption was that the effect of vehicular speed differences could best be determined by accidents involving two or more vehicles traveling in the same direction; thus head-on, single vehicle, and pedestrian accidents were not included. Speed data were submitted by the States on EAM (Electronic Accounting Machines) cards in the format shown in figure 2. The coded information represented the percentage of traffic traveling in each speed group. The data were not adjusted in any manner but were used precisely as submitted by the States.

The mean travel speed for each study unit was obtained by accumulating the products of the midspeed for each of the speed groupings—for example, 45 m.p.h. for the speed group 40–49 m.p.h.—and the percentage of the vehicles traveling within the speed group, then dividing the final total by 100. The midspeed used for the *under-40-m.p.h.* speed group was 37.5 m.p.h. for rural areas and 32.5 m.p.h. for urban areas. The midspeeds used for the *80-m.p.h.-or-more* speed group was 85 m.p.h. These midspeeds were determined from speed trend data collected on Interstate highways in many States (*4*).

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[1] The italic numbers in parentheses identify the references listed on p. 75.

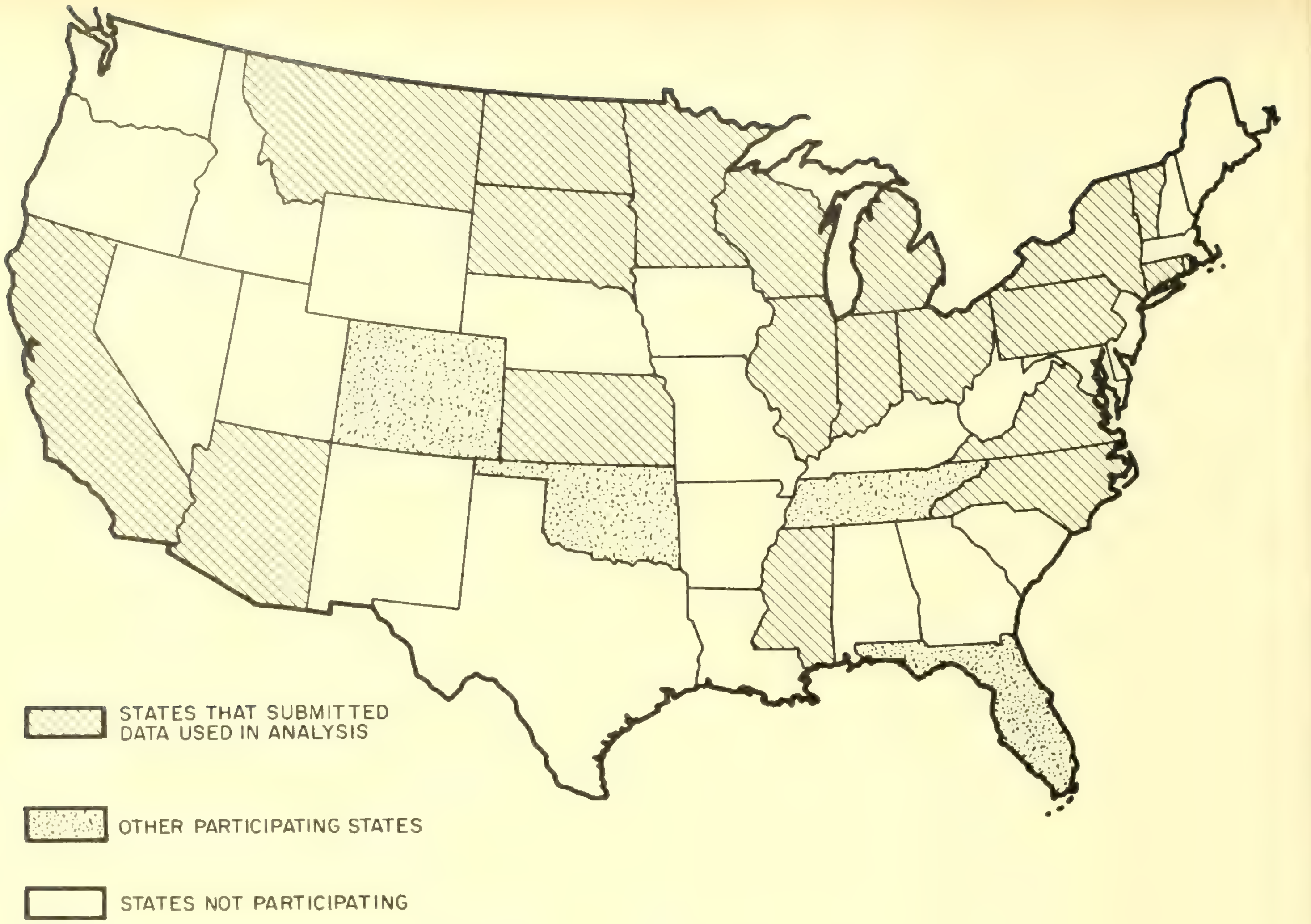
STATES THAT SUBMITTED DATA USED IN ANALYSIS
OTHER PARTICIPATING STATES
STATES NOT PARTICIPATING

Figure 1.—States participating in Interstate System Accident Research, Study II.

Results of Speed Analysis

Results of the analysis indicated that a reduction in the variation of speed among vehicles should significantly reduce accidents.

The procedure for determining involvement rates, as related to mean speed, was similar to that reported by Solomon for 2-lane and 4-lane rural highways (*2*). Involvement rate is the number of involvements per 100 million vehicle-miles and implies a vehicle involved in an accident. Thus, one accident involving three vehicles is counted as three involvements. The curve shown on figure 3 was plotted on the basis of variation from the mean speed of each unit. The involvement rate at each speed, for each study unit, was related to the variation from the mean speed of the study unit. For each accident that occurred on a study unit being used, the speed of each vehicle involved in the accident was subtracted from the mean speed of the study unit. For example, suppose the mean speed of a study unit was computed to be 60 m.p.h. and a vehicle involved in an accident on this unit was traveling at 55 m.p.h.,[2] then this involvement would be reported as having occurred at a variation of minus 5 m.p.h. from the mean speed. All such data were grouped together to obtain a data point; results of these calculations have been summarized and are shown in table 1. The data points weer plotted on figure 3, in addition to points obtained by Solomon. As these are daytime data, only Solomon's daytime curve was plotted to provide a common basis for comparison.

In table 1, it is shown that the lowest involvement rate occurred at approximately +12 m.p.h. above the mean speed of a study unit. One might expect the lowest involvement rate to occur at the mean speed; but the variation inherent in collecting and estimating speed data is possibly the reason that the lowest involvement rate occurs at +12 m.p.h. above the mean speed. However, as the magnitude of the variation increased, either above or below the mean speed, the involvement rate increased. These results were remarkably similar to those reported by Solomon. This curve is shifted slightly to the right of Solomon's curve (see fig. 3), in which the lowest involvement rate occurred at approximately +8 m.p.h. for daytime accidents; but Solomon's study was conducted on 2-lane and 4-lane main rural highways that had no control of access. Usually, on this type of conventional highway, the average speed is lower than on the Interstate highway; the mean speed was about 52 m.p.h. on conventional rural highways and 59 m.p.h. on Interstate highways.

Table 1.—Involvement rate by deviation from mean speed

Deviation from mean speed	Involvements	Vehicle-miles	Involvement rate[1]
m.p.h.	*Number*	*Millions*	*Number*
−30.0 to −34.9	82	.13	63,222
−25.0 to −29.9	129	1.93	6,673
−20.0 to −24.9	109	14.03	777
−15.0 to −19.9	245	86.86	282
−10.0 to −14.9	259	180.91	143
−5.0 to −9.9	356	519.52	68
0.0 to −4.9	321	755.41	42
+0.1 to +4.9	290	772.84	37
+5.0 to +9.9	162	566.95	28
+10.0 to +14.9	46	180.38	25
+15.0 to +19.9	21	60.13	35
+20.0 to +24.9	14	10.29	136
+25.0 to +29.9	10	3.25	307
+30.0 to +34.9	13	.11	11,627

[1] Involvement rate=number of involvements per 100 million vehicle-miles.

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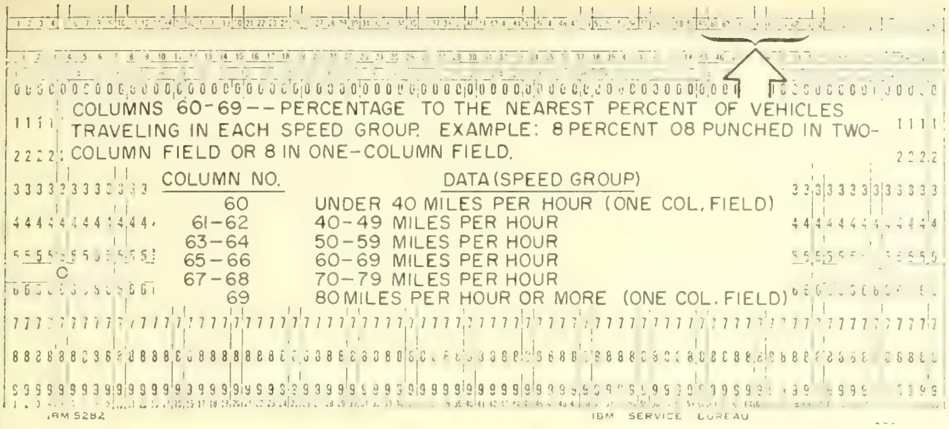
[2] Speeds submitted by the State and probably extracted from accident report forms.


COLUMNS 60-69--PERCENTAGE TO THE NEAREST PERCENT OF VEHICLES TRAVELING IN EACH SPEED GROUP. EXAMPLE: 8 PERCENT 08 PUNCHED IN TWO-COLUMN FIELD OR 8 IN ONE-COLUMN FIELD.

COLUMN NO.	DATA (SPEED GROUP)
60	UNDER 40 MILES PER HOUR (ONE COL. FIELD)
61-62	40-49 MILES PER HOUR
63-64	50-59 MILES PER HOUR
65-66	60-69 MILES PER HOUR
67-68	70-79 MILES PER HOUR
69	80 MILES PER HOUR OR MORE (ONE COL. FIELD)

Figure 2.—Speed data collected for Study II.

Of more importance is the generally lower accident rate observed on Interstate highways. Although the average speed of vehicles on the Interstate System is 7 m.p.h. higher than on conventional main rural roads, the Interstate system can better accommodate differences in vehicle speeds, with the exception of very slow-moving vehicles. It appears, however, that with respect to accident involvement on freeways, as well as on conventional rural highways, both very high and very low speeds are dangerous, and it is differences in speed among vehicles that cause hazardous situations. The hazard of slow-moving vehicles on high-speed highways is indicated by the sharp rise in involvement rate for vehicles traveling 25 m.p.h. below the mean speed.

Level of law enforcement

An attempt was made to investigate the effect of the level of law enforcement on mean speed and accident involvement. Data submitted represented the average number of oral warnings, written warnings, arrests, and police patrol hours per mile per year on the Interstate System. Only study sections for which this information was provided were used in this analysis. Law enforcement information was requested for mainline study units only, but in several States, this information was not available and these sections were not used in this portion of the analysis.

Enforcement data were collected on an average daily basis, and speed data were collected for daytime hours only. Therefore, it was assumed that the average daily enforcement data were proportional to the level of daytime enforcement. As speed data were collected for daytime hours alone, only those accidents—single and multivehicle—occurring between 9 a.m. and 4 p.m. were used. The other criteria for the data base in this analysis were the same as the criteria used in the speed analysis—that is, no distinction was made between urban or rural sections; 2-lane two-direction mainline units were eliminated; only mainline units and speed-change lanes were used, and only traffic volumes between 9 and 4 p.m. were considered.

Results shown in table 2 indicate that no trend can easily be established between an increase in warnings, arrests, or police patrol and the mean speed of travel or the involvement rate on a study section. Further investigation of enforcement related to traffic volume and other variables will be undertaken in the future.

Effect of interchanges on accident rates

In the analysis of the effect of interchanges on accident rates, all units were divided into urban or rural sections. Each mainline unit was then positioned by its proximity to an interchange. Because each unit was located between two interchanges, ahead and behind, accidents were assigned to the nearest interchange. Units equidistant from two interchanges were divided between the two interchanges.

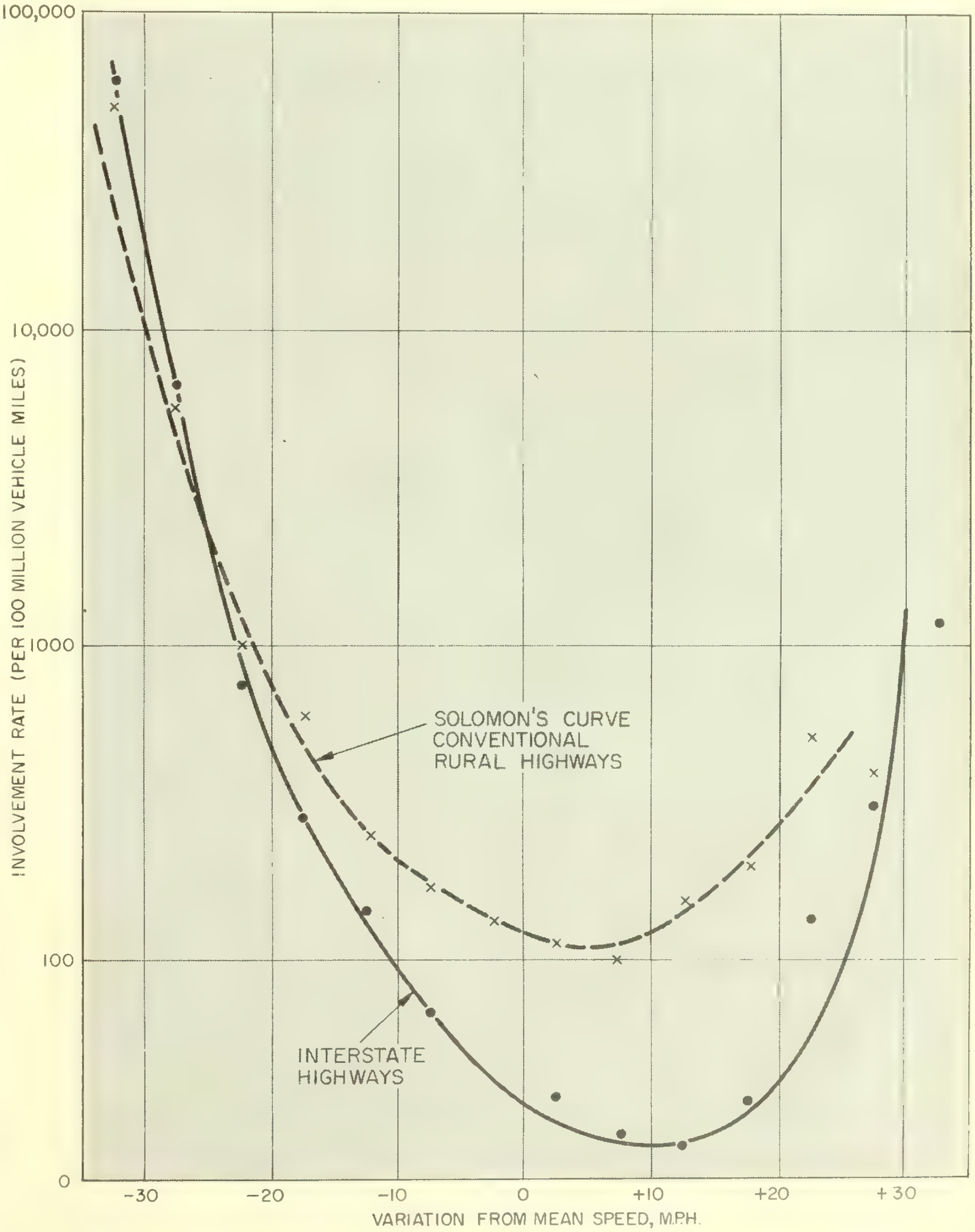


Figure 3.—Accident involvement rate by variation from mean speed on study units.

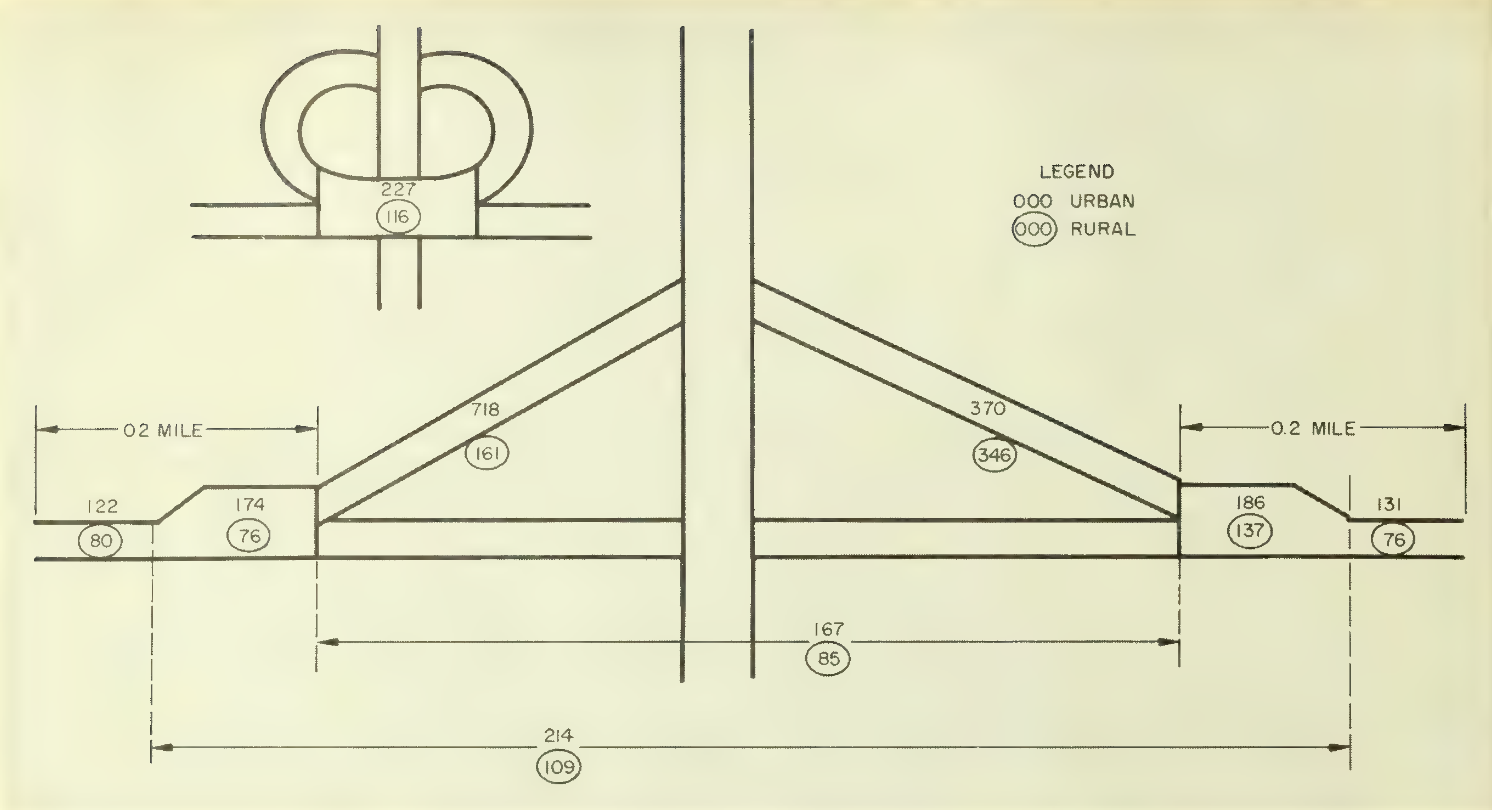


Figure 4.—Accident rate by type of interchange unit.

Distances were measured from the midpoint of each study unit to the gore (beginning of the ramp) and were recorded in discrete codes which represented continuous intervals of unequal length. The accident rates—the number of accidents per 100 million vehicle-miles—for both *between-interchange* units and *at-interchange* units were calculated. An interchange was assumed to extend from the beginning of the deceleration lane taper to the end of the acceleration lane taper. Thus, the following interchange units were included in this analysis:

- Deceleration lanes including taper
- Acceleration lanes including taper
- Exit ramps
- Entrance ramps
- Mainline units between speed-change lanes
- Combined acceleration-deceleration lanes

The *at-interchange* accident rate, shown on figure 4, was a weighted combination of the accident rates for each of these units. Accident rates were not calculated for crossroad units, terminal areas between ramps and crossroads, frontage roads, and local streets.

In interpreting the results of the analysis, it is essential to note that the only variables considered were the distances between the study unit and the interchange, and the classification of the section—rural or urban. No other variables were considered.

Table 2.—Involvement rate by type and level of enforcement

Type of enforcement	Level of enforcement per mile per year	Involvements		Mean speed	Vehicle miles		Involvement rate [1]	
		Cars	Trucks		Cars	Trucks	Cars	Trucks
	Number	*Number*	*Number*	*m.p.h.*	*Millions*	*Millions*	*Number*	*Number*
Oral warnings	Less than 5	960	124	58.7	1,856	371	52	33
	5-14	140	59	58.3	287	71	49	82
	15-34	636	125	59.0	769	173	83	72
	35-74	537	103	59.8	632	124	85	83
	75-149	458	54	57.8	398	68	115	79
	150-299	308	39	58.3	354	49	87	80
	300-549	74	9	58.0	42	6	178	154
	600-1,199	14	2	57.8	7	2	195	108
	Over 1,200 [2]	---	---	------	---	---	---	---
	Unknown	211	22	57.2	407	77	52	28
Written warnings	Less than 5	848	167	57.5	1,361	28	62	60
	5-14	423	38	59.0	591	104	72	36
	15-34	717	101	60.4	805	179	89	56
	35-74	322	72	60.2	577	127	56	57
	75-149	399	94	60.3	701	141	57	67
	150-299	422	44	60.4	426	65	99	67
	300-549	101	10	57.4	144	25	70	40
	600-1,199	31	5	59.8	71	11	44	44
	Over 1,200 [2]	---	---	------	------	---	---	---
	Unknown	75	6	50.0	77	14	98	42
Car hours of police patrol.	Less than 5	13	0	57.9	13	4	96	00
	5-14	4	2	60.5	67	12	6	16
	15-34	68	7	55.9	204	43	33	16
	35-74	140	46	59.9	27	61	51	75
	75-149	337	74	58.2	568	142	59	52
	150-299	731	195	59.3	827	190	88	102
	300-549	296	41	59.5	774	168	38	24
	600-1,199	1,210	124	58.2	1,626	269	75	46
	Over 1,200	460	41	58.4	316	39	146	104
	Unknown	79	7	51.3	90	14	88	50
Arrests	Less than 5	130	12	57.5	323	661	40	18
	5-14	201	58	58.0	367	83	55	70
	15-34	501	117	59.2	832	182	60	64
	35-74	639	162	59.4	1,263	312	51	52
	75-149	876	101	58.7	1,082	170	81	59
	150-299	594	49	58.2	528	78	112	63
	300-549	292	24	58.7	198	27	148	90
	600-1,199	67	12	60.6	26	4	254	326
	Over 1,200 [2]	---	---	-----	-----	-----	---	---
	Unknown	38	2	54.1	134	21	28	9

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[1] Involvement rate = number of involvements per 100 million vehicle-miles.
[2] No data available.

Table 3.—Accident rate by proximity to interchange ahead or behind

EXIT SIDE			ENTRANCE SIDE		
Distance to exit-ramp nose ahead	Accidents	Accident rate [1]	Distance to entrance-ramp nose behind	Accidents	Accident rate [1]
URBAN			URBAN		
	Number	*Number*		*Number*	*Number*
Less than .2 miles	722	131	Less than .2 miles	426	122
.2–.4 miles	1,209	127	.2–.4 miles	1,156	125
.5–.9 miles	786	110	.5–.9 miles	655	105
1.0–1.9 miles	280	75	1.0–1.9 miles	278	84
2.0–3.9 miles	166	63	2.0–3.9 miles	151	59
4.0–7.9 miles	[2] 19	69	4.0–7.9 miles	200	75
More than 8 miles [3]			More than 8 miles [3]		
RURAL			RURAL		
Less than .2 miles	160	76	Less than .2 miles	117	80
.2–.4 miles	459	75	.2–.4 miles	482	82
.5–.9 miles	559	69	.5–.9 miles	560	72
1.0–1.9 miles	479	69	1.0–1.9 miles	435	64
2.0–3.9 miles	222	68	2.0–3.9 miles	169	51
4.0–7.9 miles	46	62	4.0–7.9 miles	52	40
More than 8 miles [3]			More than 8 miles	-------	--------

[1] Number of accidents per 100 vehicle-miles.
[2] Small sample size.
[3] No data available.

Results

The results reported below indicate that in urban areas, proximity of a study unit to an interchange had a substantial effect on the accident rate. A similar effect, of less magnitude, was observed in rural areas for study units near entrance ramps.

Between-interchange accident rates

As shown in table 3, the accident rate decreased on urban sections as the study unit was positioned farther away from an exit ramp; the highest rate occurred in units less than 0.2 mile from the exit ramp. This decrease was substantial to a distance of approximately 2 miles from the ramp. Also, as a unit was stationed farther from the entrance ramp area, the accident rate decreased, although not uniformly. Moreover, the rates on both sides of the interchange were fairly comparable. On rural sections, however, the change in rates, as a unit was positioned closer to the interchange, was not significantly altered; and in the exit direction it remained almost constant. Thus, in urban areas proximity to interchanges seemed clearly to affect the accident rate, probably because in urban areas interchanges occur almost twice as frequently as in rural areas (table 4), and usually carry much higher volumes.

Table 4.—Interchange-mileage relations by area type

Area type	Urban	Rural
Number of interchanges	718	942
Number of miles	1,380	3,919
Interchanges per mile	.52	.29
Miles between interchanges	1.9	3.4

At-interchange accident rates

Accident rates are presented, in figure 4, for each type of *at-interchange* unit; sample size is indicated in table 5. The total *at-interchange* accident rate was, as noted above, a weighted combination of the accident rates for each separate unit type computed for the 100 million vehicle-mile base.

When interpreting the figure, it is important to note that only exit ramps and entrance ramps are shown. Included in these calculations are ramps which are part of diamond-type interchanges, outer connections and loops of cloverleaf interchanges, semidirect and direct connections, and slip ramps.

The accident rate for urban interchanges is substantially higher than for rural interchanges, as these areas carry more traffic, making merging and diverging maneuvers more difficult. Because of higher right-of-way and construction costs, urban interchanges tend to be less standard in design, are more complex, and are confined to smaller areas than rural interchanges. These factors increase conflict possibilities, and also make entrance and exit maneuvers more difficult. The exceptionally high accident rate on entrance ramps in urban areas may be caused by inadequate acceleration lanes, or the lack of them, on many sections, necessitating vehicles to stop at the bottom of the ramp before moving into the traffic stream. Also, the unavailability of sufficient gaps in the main traffic stream makes it difficult to merge into moving traffic.

The accident rate on the mainline decreases after the deceleration lane has been passed (figure 4). It appears that after the decision point at the deceleration lane has been passed, the chances of an accident decrease.

From this brief analysis it can be determined that sections in proximity of interchanges experience a higher accident rate than other sections. Ramps have much higher accident rates than speed-change lanes (and paralleling main roadway) and these, in turn, have generally higher rates than the other portions of the main roadway.

Conclusion

The results reported demonstrate that on the Interstate System, as the speed of a vehicle varies from the mean speed of traffic, either above or below the mean speed, the chance of the vehicle being involved in an accident increases; that the level of enforcement has little or no apparent effect on the mean speed or on the accident experience of a study section; and that proximity to interchanges, especially in urban areas, appears to affect significantly the accident experience of the study section.

Although these results are not conclusive they provide some insight into areas in which more intensive research should be conducted, such as interchange spacing and utilization and more effective methods of speed control.

Table 5.—Accident rate by interchange unit and area type

Interchange unit	Area type					
	Rural			Urban		
	Vehicle-miles *100 Million*	Accidents *Number*	Accident rate [1] *Number*	Vehicle-miles *100 Million*	Accidents *Number*	Accident rate [1] *Number*
Deceleration lane	2.51	348	137	5.83	1,089	186
Exit ramp	0.57	199	346	1.48	546	370
Area between speed change lanes	6.52	554	85	11.87	1,982	167
Entrance ramp	0.59	95	161	1.61	1,159	719
Acceleration lane	3.68	280	76	8.40	1,461	174
Acceleration-deceleration lane	0.49	87	116	2.45	555	227
Total	14.36	1,563	109	31.64	6,792	214

[1] Accident rate = number of accidents per 100 million vehicle-miles.

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(3) *The Lone Driver Involved in Injury-Producing Accidents*, by William E. Schotz, Cornell Aeronautical Laboratory Inc., Cornell University, Buffalo, N.Y., Apr. 25, 1966.

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NEW PUBLICATION

Standard Plans for Highway Bridges, vol. II, Structural Steel Superstructures, 1968 ($1.00 a copy), is a revision of the 1962 edition with respect to bridge widths and current design specifications. These plans are intended to serve as a useful guide in the development of suitable and economical bridge designs. An effort has been made to give sufficient information on all plans so that they may be readily modified in the preparation of contract drawings.

The volume contains plans for simple span I-beam and welded girder superstructures from 20 feet to 180 feet, simple span two-girder with floor system superstructures from 120 feet to 200 feet, and continuous 3-span I-beam and welded girder superstructures with center spans from 50 feet to 240 feet. Bridge roadway widths used are 28 feet with H15–44 live load for low traffic volume low design speed roadways, 44 feet with HS20–4 live load for the standard 2-lane two-dire tional roadway and 40 feet with HS20–44 li load for the standard 2-lane one-direction roadway.

One series of simple span I-beam supe structures with Interstate loading and variable width roadway is included in th plans.

Passing Aid System I, Initial Experiments

(*Continued from p. 70*)

ACKNOWLEDGMENTS

The cooperation of many persons of the Public Roads staff was necessary for the test driving and the collection and analysis of data for the experiments of Passing Aid System I. The persons assisting with the collection of field data included Raymond Greenwell, Santo Salvatore, Howard S. Ellis, and John Porter. The electronics system, designed by Raleigh Emery, was kept operational during driver test runs by Messrs. Novean and Porter. Margaret Cormack assisted with the scheduling of test subjects, the administrative details of ordering equipment, and the preparation of the final report. Statistical aid was supplied by Dr. Harry Weingarten and Mrs. Phyllis Mattison. Mr. David Solomon provided overall guidance and optimistic enthusiasm in conducting the analysis of Passing Aid System I.

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Federal-Aid Highway Map (42 x 65 inches) (1965). $1.50.

Federal Laws, Regulations, and Other Material Relating to Highways (1965). $1.50.

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Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354, 88th Cong. 2d sess. (1964). 45 cents.

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Vol. IV—Typical Continuous Bridges (1962). $1.00.

Vol. V—Typical Pedestrian Bridges (1962). $1.75.

Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.

Traffic Assignment Manual (1964). $1.50.

(1963). 15 cents.

Traffic Safety Services Directory of National Organizations

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A JOURNAL OF HIGHWAY RESEARCH

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

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Criteria for Highway Tunnel Lighting Design

Tunnel lighting in Thimble Shoal Tunnel, Chesapeake Bay Bridge and Tunnel District, Norfolk, Va.

BY THE OFFICE OF
ENGINEERING AND OPERATIONS
BUREAU OF PUBLIC ROADS

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Modern highways designed to accommodate high-speed, high-volume vehicular traffic incorporate in their construction numerous tunnels and underpasses. Many of these structures require fixed lighting to provide adequate visibility for the road user. Information obtained during a national survey of the lighting system in most of the major tunnels throughout the United States is presented in this article, and the authors discuss the important lighting features noted during the survey.

Suggestions are offered for determining which tunnels and underpasses should be lighted; factors which must be considered by the designer when selecting lighting criteria for a specific project are outlined; and guidelines for both day and night levels of illumination in these facilities are summarized.

Introduction

THE INCREASING use of the automobile today, the higher standards of highway design, and the projected expansion of the Nation's highway construction programs are creating a substantial increase in the number of underpasses and tunnels both in urban and rural areas. In urban areas, the increase is necessitated by esthetic considerations in highway construction, urban planning philosophies, and the use of air rights above the highway for building construction; in rural areas, it is brought about by improved highway design standards and construction methods.

The increased number of tunnels and underpasses, together with higher vehicle operating speeds, has increased the need for proper lighting of these structures. The attention being given highway safety emphasizes this need, not only for the safety of vehicle occupants and pedestrians but also to insure the efficient use of the highways.

How to distinguish an underpass from a tunnel and when to provide lighting for structures between 75 feet and 1,000 feet long are questions that need clarification. It is likely that any guidelines or standards for tunnel lighting may not be universally applicable and that each of the structures should be individually evaluated.

Once a structure is defined as a tunnel, such factors as length, cross-section, lining, traffic volume, alinement and grades, location, and outside field-of-view brightness immediately adjacent to the portal should be considered in the design of the lighting. But so many combinations of variables can exist that no two tunnels are alike, and even though tunnel lighting guides are available (*1, 2, 3*),[2] an onsite evaluation of existing lighting systems, to determine fundamental objectives and operational experience, is a necessary prerequisite to an evaluation of tunnel lighting.

In this article, a library study of literature on highway tunnel lighting and a field investigation of existing tunnels are summarized. The three major objectives of the research were (1) to identify published materials related to the lighting of tunnels, (2) to summarize the survey of existing tunnel lighting installations, and (3) to distinguish between a tunnel and an underpass for lighting purposes and formulate guidelines for illumination, based on pertinent research and the field investigation.

For the first of these objectives, studies were conducted in Washington, D.C., at the Library of Congress, the library of the Catholic University of America, and the professional library of the Bureau of Public Roads. Night Visibility Bulletins of the Highway Research Information Service and reports of the Highway Research Board were reviewed, in addition to industry bulletins, publications, and equipment specifications on tunnel and underpass lighting solicited from manufacturers.

For the second objective, information was obtained directly from officials responsible for the design, operation, and maintenance of the major tunnels throughout the United States. A comprehensive questionnaire, submitted to the officials, consisted of nine parts:

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[1] Messrs. Thompson and Fansler were formerly in the Office of Engineering and Operations, Highway Standards and Design Division.

[2] Italic numbers in parentheses refer to the bibliography listed on p. 105.

- Tunnel physical data
- Portals and approaches
- Traffic conditions
- Electrical system
- Lighting system—continuous burning
- Supplemental daytime entrance (adaptation) lighting
- Maintenance procedures
- Cost information
- Miscellaneous

The questionnaire was mailed before the tunnel was officially visited for an on-site (day and night) inspection of tunnel lighting, operation, and maintenance, and for interviews with design, operation, and maintenance personnel to obtain their evaluations of the lighting installations. In general, complete information was furnished in all but the cost item of the questionnaire. In many of the tunnels, the initial cost of the lighting was included with other work and was not available. A summary of the tunnel lighting survey is given in table 1.

For the third objective the published literature, questionnaires, field inspections, and interviews were analyzed to identify criteria for determining the difference between an underpass and a tunnel for lighting purposes, and to determine criteria for proper highway tunnel lighting design.

Lighting terminology for a tunnel is shown in figure 1.

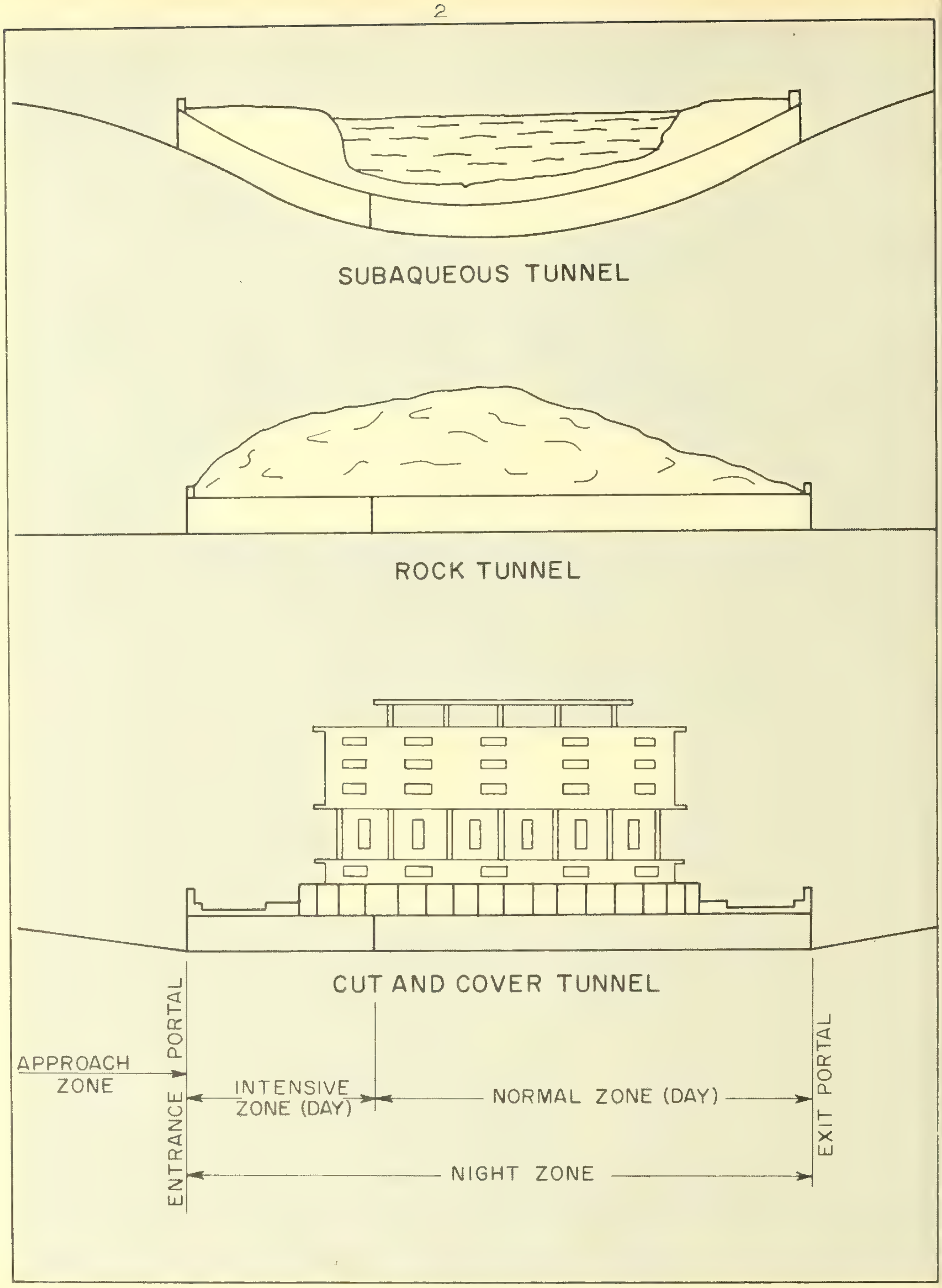

Figure 1.—Tunnel lighting terminology.

Background

Good visibility is a prerequisite to safe traffic operation in a highway tunnel. The lighting in a highway tunnel has too often been designed on the basis of insufficient guidance and opinion rather than on a factual basis. Upon completion of the nationwide tunnel lighting survey, the general observation of day and night lighting levels indicated an excess of nighttime lighting and a deficiency of daytime entrance-zone lighting. When either of these conditions exists, highway safety and highway utility are not being served to the best interest of the road user. Examples of well designed night lighting and of well designed daytime lighting were observed, but a combination of well designed day and night lighting was not found in any one tunnel. In many tunnels inadequate day lighting existed because the same type of equipment was being used both for night and day conditions. With the lighting equipment available at present, it may be impractical to use the same equipment for both day and night conditions because of the difference in the lighting requirements.

Many illuminating engineers are convinced that the foremost shortcoming in the field of tunnel lighting is the lack of basic research to isolate visibility requirements, and the instrumentation to check these requirements. Others believe that new research would result, largely in the rediscovery of many old truths; they are convinced that to reveal and discredit many ideas, allegedly motivated by commercial interest, would be rewarding. This group does not agree that there is a lack of basic research and that the gap between available research and needed information is small. The research reported here revealed that little attention has been devoted to the lighting of tunnels, and that technical advances and research on the subject have not been recognized, or remain unknown to those who are responsible for the tunnel lighting design. Evaluations of tunnel lighting by such individuals often reveal that the criteria used and the judgments exercised in design and operation continue to be clouded with personal, and frequently inept, opinion. Therefore, analysis of available information, and its dissemination to those responsible for making final design decisions, is essential to the proper consideration of sound engineering judgment and fiscal aspects.

Tunnel lighting design should account for the most important aspects of lighting—the road user's ability to see. Eye adaptation is a phenomenon that requires understanding to evaluate this ability. Eye adaptation is much faster when the eye transfers from a dark to bright environment than when it transfers from bright to dark environments. Accordingly, supplemental daytime entrance lighting is critical, whereas supplemental daytime exit lighting is not. At night, the entrance lighting becomes less critical, but attention should be given to exit lighting because of the adaptation of the driver's eye from light in the tunnel to the darker environment outside the tunnel.

During the day, the entrance of the tunnel requires sufficient lighting to prevent the effect of entering a *dark hole*, the most serious demand on eye adaptation. The exit of the tunnel appears as a *light hole* in the daytime and presents fewer problems. At night, the *dark hole* may be at the exit and the *light hole* at the entrance—a situation created by too much light in the tunnel. The survey of existing tunnel lighting showed this condition to be the rule rather than the exception.

Research and technical advancements have not made momentous strides, but significant contributions that provide ways to greatly improve tunnel lighting have been made.

In the tunnel lighting guides (*2*, *3*), it is recommended that daytime entrance lighting be provided for 15-second eye-adaptation time from the instant that the road user enters the tunnel. Based on studies of California tunnels, this adaptation period exceeds road user requirements (*4*). With the 6-foot-lambert wall brightness of the Park Presidio Tunnel in San Francisco, full iris and partial retinal adaptation times varied among individuals, but no more than 2 seconds were required for satisfactory adaptation in this tunnel. At the Posey Tunnel in San Francisco, which had a low interior brightness ranging from 0.03 to 0.5 footlamberts, satisfactory adaptation occurred on a bright day within 3 or 4 seconds after the tunnel was entered, although visibility remained relatively poor. It was concluded from these studies that long stages of daytime adaptation lighting are wasteful, that visibility of obstacles within the tunnel is poorest before the tunnel is entered, and that daytime entrance lighting is needed to provide better visibility before entering.

An understanding of certain characteristics of the human eye is important to establish criteria for tunnel lighting design. It is sometimes thought that changes in pupil size are important in allowing the eye to work efficiently over a wide range of light intensities, but, actually, the pupil area changes over a range of brightness of only about 16 to 1, whereas the eye works efficiently over a range of about 100,000 to 1 (*5*). In a 1943 study, it was established that the response of the small foveal area of the eye's retina accounts for approximately 90 percent of the adaptation to a uniform field (*6*). As shown in figure 2, the fovea is a depression at the back of the retina, the point at which the vision is most acute, corresponding to approximately the central 2 degrees of the human visual field. Complete foveal vision adaptation requires about 7 minutes; complete peripheral (or retinal) vision adaptation continues for an hour or more (*5*). The adaptation time for tunnel lighting may be considerably shorter than the time necessary to reach the final state of adaptation.

In a 1916 study, it was established that, in normal eyes, the pupil contracts automatically to protect the retina from excessive brightness, and expands to admit light to the retina when it is required (*7*). On sudden exposure to a bright light, the diameter of the pupil varies from about 2 to 8 millimeters, contracting in about 1 second. On a sudden reduction of full brightness, the pupil dilates to about 5 millimeters in about 2 seconds, and then continues to expand slowly to its limit.

In a 1920 study (*8*) concerning the rate of change of pupil size for instantaneous changes in image brightness, it was shown that only a fraction of a second is required for the pupil to expand—increase its area 2½ times—after a scene was changed from 1,000 footlamberts to 30 footlamberts.

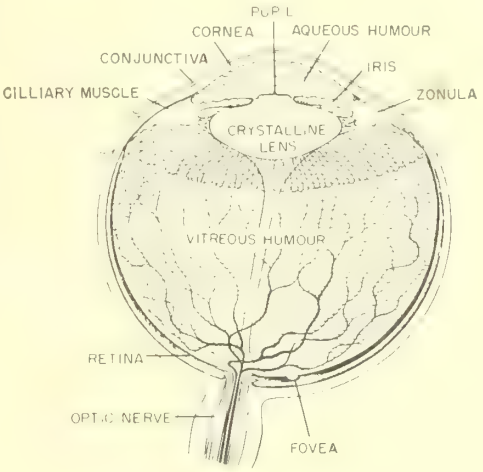

Figure 2.—The human eye.

Complete adaptation is not necessary for good visibility. Partial adaptation is the objective, and there is sufficient evidence to support a lighting design that provides an adaptation time of only 4 seconds, which even exceeds the minimal road user requirements. Visibility must be provided for the road user outside the tunnel, or just about to enter, to see a stalled vehicle or smaller object inside the tunnel. However, the principal problem is not the rate of adaptation, which takes place very quickly, but rather the adaptation level of a road user approaching the tunnel entrance (*4*, *9*). The problem of adaptation level is aggravated by the effects of atmospheric scatter of the aerosals (haze) between the road user and the tunnel portal. In bright sunlight this haze, composed of dust and vehicle exhaust, can produce an apparent brightness at the portal of above 100 footlamberts and is a hindrance to visibility (*9*).

The visibility required for the road user to see objects within the tunnel as he approaches the entrance can be achieved by only two methods: (1) by reducing the adaptation level of the road user; and (2) by greatly increasing the brightness of the background within the tunnel against which objects can be seen (*9*).

Roadway brightness has less effect on the road user's visibility than wall and ceiling brightness, except in wide tunnels (*10*). Generally, roadway brightness can be assigned a secondary consideration in the lighting design because with sufficient wall and ceiling brightness, adequate roadway brightness will automatically follow (*4*).

Because the tunnel walls and ceiling constitute a background to silhouette a vehicle to the road user outside the tunnel, or just about to enter it, a high level of illumination is required to provide high brightness of these surfaces. If the tunnel is straight and level, this intensive lighting zone must extend far enough into the tunnel to provide a background for vehicles, perhaps beyond what is normally required. If the tunnel has curved horizontal or vertical alinement near the entrance, the intensive lighting design is much simpler.

One of the worst conditions for tunnel lighting may occur when the tunnel portal faces the south—a condition for which there is little hope of achieving sufficient tunnel wall and ceiling brightness by conventional lighting systems of reasonable magnitude (*11*). This condition can be improved considerably by keeping the reflection of the portal and surrounding area low, thus reducing the brightness level in the motorist's field of view. Also, the brightness in the road user's field of view can be reduced by roadway louvers or screens located outside the tunnel entrance.

In summary, the visual requirements of the road user are most critical when he is still outside and immediately entering the tunnel, not after he has entered it. When he leaves the tunnel, the visual requirement isn't acute. The entrance problem is basically range of adaptation, not rate of adaptation. There are two possible ways to overcome this problem: Either provide enough brightness on the inside of the tunnel, or design the approach to the tunnel so that the brightness level in the road user's field of view is reduced. In good design, both methods should be evaluated so that they can be considered in combination.

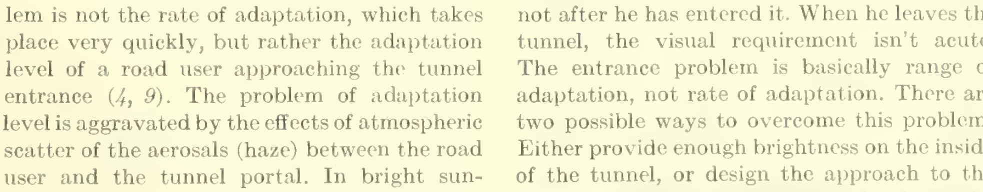

Figure 3.—Underpass dark-frame and obstacle visibility.

Table 1.—Summary of tunnel lighting survey

Name	Location	Physical data						
		Length	Year built	Type pavement	Tubes			
					No.	Width		Operation
		ft.				*ft.*	*in.*	
John F. Fitzgerald Expressway.[2]	Boston, Mass	2,335	1958	Portland cement concrete	2	40	0	1-way
Wm. F. Callahan, Jr.	do	5,047	1961	Asphaltic concrete	1	26	9	do
Prudential Passageway.	do	1,980	1965	Portland cement concrete	2	49	0	do
Lincoln Tunnel (3d tube)	New York, N.Y	8,013	1959	Asphaltic concrete	1 (of 3)	26	4	do
Holland Tunnel	do	8,557	1927		2	25	0	do
George Washington Bridge Lower Level	do	600 eastbound; 800 westbound.	1962		2	42	0	do
Fort Pitt	Pittsburgh, Pa	3,550	1960	Brick	2	29	4	do
Allegheny No. 2 (New)	Pennsylvania Turnpike	6,070	1964	Asphaltic concrete	1	31	9	
Hampton Roads	Norfolk, Va	7,479	1957	do	1	27	0	2-way
Norfolk-Portsmouth No. 2 (Midtown)	do	4,194	1962	do	1	28	10	do
Thimble Shoal Tunnel	Chesapeake Bay Crossing, Norfolk, Va.	5,738	1964	do	1	28	9	do
Baytown Tunnel	Baytown, Harris County, Tex	3,009		Portland cement concrete	1	26	10	do
State Highway No. 183 Tunnel	SW International Airport, Fort Worth, Tex.	813	1964	do	2	48	0	1-way
Caldecott Tunnel No. 3 (New)	Oakland, Calif	3, 371	1964	do	1	34	6	do
Webster St. Tube	Almeda-Oakland, Calif	3,340	1962	Asphaltic concrete	1	32	9	do
Waldo Tunnel Route No. 101	Marin County, Calif	1,000	1955	do	2	46 (new) 46 (old)	6 8	do
Park Presidio Tunnel	San Francisco, Calif	1,300		Portland cement concrete	1	49	0	2-way
Bothell Tunnel	Seattle, Wash	390	1963	do	2	32	0	1-way
Roanoke Tunnel	do	662	1963	do	1	25	0	do

Name	Location	Night lighting (normal day zone)					
		Year lighting installed	Lamp data		Luminaire		
			Type	Current	Data	Location	Spacing
				Milliamperes			
John F. Fitzgerald Expressway.[2]	Boston, Mass.	1959	Fluorescent 72-in. T8 Slimline.	120	Lamp in pyrex tube	Ceiling near each wall.	1 row continuous on each side.
Wm. F. Callahan, Jr	do	1961	do	150	do	Walls approx. 12 ft. above roadway.	do
Prudential Passageway	do	1965	Fluorescent 96-in. T12/HO.	800	8-ft. 3-lamp internal reflector.	Wall, 15 ft. above roadway.	do
Lincoln Tunnel (3d tube)	New York, N.Y	1959	Fluorescent [4] 72-in. T12 Slimline.	325–600	Lamp in pyrex tube	Wall near ceiling	do
Holland Tunnel	do	1963 [5] (Relighted)	Fluorescent 96-in. T12 Slimline.	430	do	1 row on ceiling over safety walk and other on upper wall surface.	do
George Washington Bridge Lower Level	do	1962	Fluorescent 72-in. T12 Slimline.	350	do	Wall approx. 1 ft. below ceiling.	do
Fort Pitt	Pittsburgh, Pa	1960	do	200	do	Ceiling near wall	do
Allegheny No. 2 (New)	Pennsylvania Turnpike.	1964	Fluorescent 96-in. T12/HO.	1, 000	Internal reflector 1 HO lamp operated in each 8-ft. luminaire.	Wall near ceiling	do
Hampton Roads	Norfolk, Va	1957	Fluorescent 72-in. T8 Slimline.	180	Lamp in pyrex tube	Wall 12 ft. below ceiling.	do
Norfolk-Portsmouth No. 2 (Midtown).	do	1962	do		do	Wall near ceiling	do
Thimble Shoal Tunnel	Chesapeake Bay Crossing, Norfolk, Va.	1964	do	Approximately 125	do	Wall just below ceiling.	do
Baytown Tunnel	Baytown, Harris County, Tex.	1953	Fluorescent 72-in. T12 Slimline.	200	Single lamp 6-ft. luminaire internal reflector.	Center of arch ceiling.	2 rows continuous.
State Highway No. 183 Tunnel	SW International Airport, Fort Worth, Tex.	1964	Fluorescent	1, 000	2 lamps with reflector (only 1 lamp at night).	Ceiling at junction of each wall.	do
Caldecott Tunnel No. 3 (New)	Oakland, Calif	1964	Fluorescent 96-in. T12 RS XHO (high temperature).	60	3 lamps with reflector, deflectors (only 1 lamp used).	On each side near ceiling.	do
Webster St. Tube	Almeda-Oakland, Calif	1962	do	60	do	do	do
Waldo Tunnel Route No. 101	Marin County, Calif	1955	Fluorescent 72-in. T8 Slimline.	120	3-lamp luminaires with reflector (only 1 lamp used).	On each side of both bores.	do
Park Presidio Tunnel	San Francisco, Calif	1955 (Relighted)	do	120	do	On each side near ceiling.	do
Bothell Tunnel	Seattle, Wash	1963	Fluorescent 96-in. T12 Slimline.	425	3-lamp luminaire with internal reflector.	On each wall near ceiling.	Continuous each side. Spacing 24 ft. at night.
Roanoke Tunnel	do	1963	do	430	3 lamps with internal reflector (1 lamp in each 3d luminaire used).	On each side near ceiling.	do

[1] ADT—Average Daily Traffic.

[2] Entrance and exit ramps within tunnel. Lane changes permitted.

[3] Circuitry arranged so lighting levels can be controlled. All lights now being operated during day and night. Operation being studied to determine most effective conditions.

[4] Operated 600 ma. day and night.

[5] Original lighting incandescent. New lighting used old wall boxes.

[6] Lane changes permitted.

[7] No intensive lighting on westbound tube.

Table 1.—Summary of tunnel lighting survey—Continued

Physical data—Continued

Traffic lanes each tube			Ceiling					Finish		Direction portal faces	Volume	Speed
No.	Width		Ht.		Shape	Portal	Taper	Ceiling	Walls			
	ft.	*in.*	*ft.*	*in.*		*ft.*	*ft.*				*ADT*[1]	*m.p.h.*
3	12	5	14	3	Flat			Concrete	Ivory and yellow tile	N-S	100,000	30-55
3	10	9	13	6	do			Yellow tile	Yellow tile	SW	23,000	20-35
4	11	0	14	0	do			Painted concrete	Painted concrete	E-W		55
2	10	9	13	0	do	29	130	White tile	White tile	S	27,000	35
2	10	0	13	6	do			Concrete	do	W-NE	60,000	35
[6] 3	12	0	18	0	do	28	75	White tile	Yellow tile	E-W	23,000	45-55
2	12	0	12	0				do	White tile	NE-SW	61,000	45
2	13	0	13	0				Painted concrete[9]	do	E	30,000	50
2	11	6	11	6				White metal panels	do	SE-NW	10,000	40-45
2	11	0	11	0				do	do	E W	11,000	35-45
2	12	0	14	0	Flat	[10] 19	17	Ivory tile	Ivory tile	NE SW	3,000 to 7,000	45
2	11	0	19 17	18 1	Arched			Tile	Tile	NE SW	+12,350	15 35
3	[11] 12	0	21 15	7 9	do			Painted concrete	Painted concrete	E-W		70
2	14	0	18	4	Flat			Green tile	Green tile	NE	26,000	55
2	12	0	15	0	do			do	do	NE	17,550	35
[6] 3	12	0	29	0	Arched			Yellow tile (new); concrete (old).	Yellow tile	N-S	35,000	50
[6] 4	11	0	31	0	do			Concrete	Painted concrete	N-S	54,000	45
[6 11] 2	12	0	16 (max.) 4 (min.)	4 10				Yellow tile	Yellow tile	NE-SW	14,000	30-35
1	14	0	16 (max.) 14 (min.)	5 11				do	do	N	8,800	35-40

Night lighting—Continued		Daytime entrance lighting (intensive zone)						
Roadway illumination	Approach roadway illumination	Length	Lamp data		Luminaire			Roadway illumination
			Type	Current	Data	Location	Spacing	
Average ft.-c		*ft.*		*Milliamperes*				*Average ft.-c*
3.5	400-w. mercury; 0.9-1.0 ft.-c.	Approximately 650.	Fluorescent 72-in. T8 Slimline.	300	Lamp in pyrex tube	Ceiling near wall	2 rows continuous each side.	9.0.
	400-w. mercury; 1.0 ft.-c.	1,700	do	450	do	Side walls	1 row continuous each side.	
Approximately 40.	400-w. mercury 1.2 ft.-c.						[3]	Approximately 40.
6.0	1-2 ft.-c.	800	Fluorescent[4] 72-in. T12 Slimline.	600	Lamp in pyrex tube.	Wall near ceiling	1 row continuous each side.	6.0.
5.0-6.0	Mercury 2-3 ft.-c.	250	Mercury 400 w.		R-60 reflector lamps	Recessed in each wall and over roadway.	Approximately 16 ft.	70.
7.0	Mercury 3.5-3.7 ft.-c.	250[7] east-bound only.	do		Refractor type	Recessed in ceiling 3 ft. 6 in. from wall.	First 150 ft.—5-ft. centers. Next 100 ft.—10-ft. centers.	75 (first 150 ft.); 45 (next 100 ft.)
	400-w. mercury		Fluorescent 72-in. T12 Slimline.					
20	400-w. mercury; 1.8 ft.-c.	1,800	2 96-in. fluorescent PG 17 (first 900 ft.); 2 96-in. T12/HO (next 900 ft.).	1,000	2 lamps, internal reflector.	Wall near ceiling	1 row continuous each side.	60 (first 900 ft.); 40 (next 900 ft.).
4.0-5.0	400-w. mercury; 1.0 ft.-c.	675	Fluorescent 72-in. T8 Slimline.	180-300			2 rows continuous each side.	24 (first 225 ft.); 12 (next 225 ft.); 8 (next 225 ft.).
	400-w. mercury	430	do				do	
Approximately 8-9.	Fluorescent 6.0 ft.-c.	1,459	do	125-450	Lamp in pyrex tube.	Wall near ceiling	do	Variable manually.
7-8.4	400-w. mercury; 1.0 ft.-c.	325	do	400	Single lamp with internal reflector.	Center of ceiling	6 rows continuous.	
12.0	Fluorescent 3.0	550	Fluorescent	1,000	2 lamps with reflector.	Ceiling near wall	do[12]	
Walls—5.0; roadway—3.0.	400-W. mercury; 1.0 ft.-c.	304	Fluorescent 96-in. T12 RS/XHO High temperature.	1,400	3 lamps with reflector and deflectors.	Each side	1 row continuous	Roadway—60; walls—90.
do	11,600-lumen fluorescent.	340	do	1,400	3 lamps with reflector and deflectors.	Each side near ceiling.	do	Do.
Walls—3.0; roadway—5.0.	10,000-lumen incandescent; 0.5 ft.-c.	1,000	Fluorescent 72-in. T12 RS/HO, 2 lamps.	1,000	3 lamps with reflector.	do	2 rows continuous.	Roadway—30; walls-20.
Walls—1.5; roadway—2.5.	175-w. mercury; 0.2 ft.-c.	1,300	do	1,000	do	do	do	
8.0	Mercury 1.0 ft.-c.	150	Fluorescent 96-in. T12 RS/HO, 2 lamps.	800	do	Each wall near ceiling.	do	Roadway—75-30; walls—50-34.
9-0	Mercury 0.75 ft.-c.	140	Fluorescent 96-in. T12.	800	3 lamps	do	do	Roadway—95-75 in zone 1; 65-60 in zone 2.

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[8] Lamp output changed by manual variation of current through lamps.

[9] Dull brick on face at portal.

[10] NE portal has metal sun shields above approach road for distance of 500 ft.

[11] 8-ft. shoulder on right, 4-ft. shoulder on left.

[12] First 225 ft.—3 rows each side=65; next 225 ft.—2 rows each side=40; balance—1 row each side=25.

[13] Control circuit changes operation of one lamp in each luminaire as follows: Intensive zone: Daytime 1,400 ma., Night 60 ma; Normal day zone: Daytime 600 ma., night 60 ma.

[14] Inside lane part of reversible roadway.

Definitions of Terms

Adaptation.—The process by which the retina of the eye becomes accustomed to more or less light than it was exposed to during an immediately preceding period. It changes the sensitivity of the photoreceptors to light.

Brightness (Luminance). The luminous flux per unit of projected area, per unit solid angle, either leaving a surface at a given point in a given direction or arriving at a given point from a given direction; the luminous intensity of a surface in a given direction per unit of projected area of the surface as viewed from that direction.

Footcandle.—The illumination on a surface 1 square foot in area on which is distributed a light output of 1 lumen. It equals 1 lumen per square foot.

Footlambert.—The unit of photometric brightness (luminance). A theoretically perfect surface reflecting light at the rate of 1 lumen per square foot would have a brightness of 1 footlambert.

Glare.—The sensation produced by brightnesses within the visual field that are sufficiently more intense than the luminance to which the eyes are adapted to cause annoyance, discomfort, or loss in visual performance and visibility.

Lamp.—The light source employed.

Lamp Lumen Depreciation Factor.—The multiplier to be used in illumination calculations to relate the initial rated output of light sources to the anticipated minimum rated output based on the relamping program to be used.

Luminaire Dirt Depreciation Factor.—The multiplier to be used in illumination calculations to relate the initial illumination provided by clean, new luminaires to the reduced illumination provided because of dirt collection on the luminaires at the time it is anticipated that cleaning will be required.

Louver.—A series of baffles used to shield a source from view at certain angles or to absorb unwanted light.

Lumen.—The unit of measure of the quantity of light. The amount of light that falls on an area of 1 square foot, every point of which is 1 foot from a source of 1 candela (candle).

Luminaire.—A complete lighting device consisting of a light source together with its direct appurtenances.

Maintenance Factor.—The product of the lamp lumen depreciation factor and the luminaire dirt depreciation factor.

Matte Surface.—A surface from which the reflection is predominantly diffuse, with or without a negligible specular component.

Reflectance.—The ratio of the flux reflected by a surface or medium to the incident flux. This general term may be restricted by the use of one or more of the following adjectives: regular (specular), diffuse, spectral.

Spacing.—The distance, in feet, between successive luminaires measured along the center line of the roadway.

Visual Task.—Those details and objects that must be seen for the performance of a given activity, including the immediate background of the details or objects.

Survey of Existing Lighting Installations

Underpasses

Lighting requirements of a tunnel are different from those of an underpass; and the criteria for lighting will determine whether it will be classified as a tunnel or an underpass. Consequently, to establish that a structure is a tunnel or an underpass is a prerequisite to determining the criteria for highway tunnel lighting design.

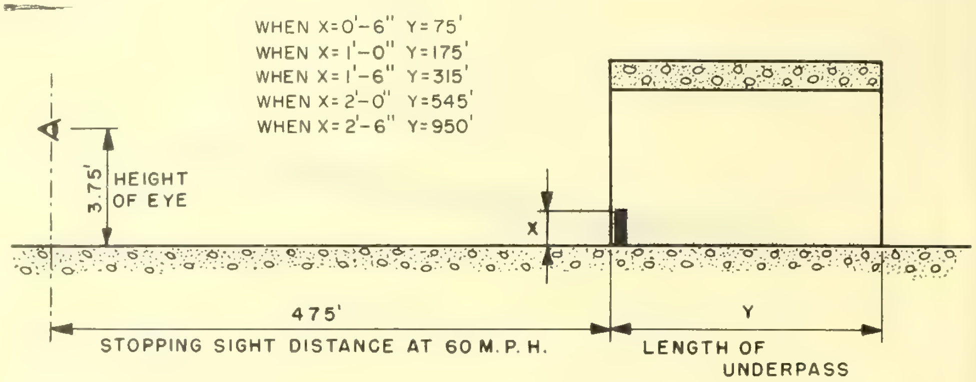

Figure 4.—Vertical dimensions for dark frame (x) across roadway for corresponding underpass lengths (y).

Unlike the lighting on a highway where warrants and justifications are considered and evaluated to determine the need, the lighting in a tunnel is as important a requirement as the ventilation or pavement surface. Daytime lighting is not required in an underpass and the warrants for night lighting would be included in the warrants for that particular section of highway of which the underpass is a part. Daytime and night lighting are required in all tunnels.

Underpasses were included in the study reported here to properly identify them so that they can be distinguished from tunnels. Several factors, including length, height, width, alinement, and grade, must be considered in determining whether a structure is an underpass or a tunnel. In daytime, as a driver approaches an underpass on a straight and level roadway, it will appear in the field of vision as a *dark frame.* After entering the underpass, the central part of the field of vision is taken up by the brightness of the exit, so that the lower brightness of the walls and ceiling has insignificant adaptation stimuli (*6*). Visibility of an object in the *dark frame* of the underpass, as shown in figure 3, is limited because the outside brightness through the underpass exit—the light center of the *dark frame*—controls the adaptation stimuli. Problems such as the influence of flicker and wall brightness, which are important in tunnels, are less important in underpasses.

Obstructions in the underpass can be seen if they are high enough to silhouette against the bright environment of the exit. If the vertical dimension of the *dark frame* across the roadway is less than the height of the smallest object that must be seen, the object will be silhouetted against the exit brightness and be visible. In figure 4 is shown the vertical dimension for the *dark frame* across the roadway and the corresponding underpass lengths, with driver's eye at a height of 3.75 feet and at a distance of 475 feet (stopping sight distance at 60 m.p.h.) from the underpass portal. The figures in the illustration apply to an underpass located on a straight and level roadway.

Because of the daylight penetration from both portals, an underpass on a straight and level roadway can be as much as 75 feet long and not require lighting during the day for an obstacle height of 6 inches. As the length of the structure is increased, the lateral dimensions of the frame created by the walls influence the silhouetting of objects within the structure. Silhouetting can be enhanced by lining the structure wall with a light colored material to reduce the *dark frame* formed by the walls.

Lighting requirements for a long underpass on a straight and level roadway, one so long that the exit provides too small a part of the field of vision to serve as an effective background, can frequently be resolved by lining the wall with a light-colored matte-finish material to better utilize natural light penetration. For such a structure, artificial light must be intense enough to compete with the natural daylight from either end of the structure. Illumination to produce the required brightness may be economically impractical, and in many structures impossible to achieve (*10*). A substantially lower level of illumination is frequently installed in this type of

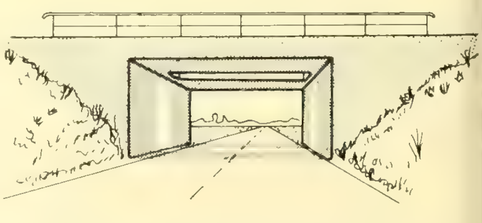

Figure 5.—Effect of ceiling opening approximately midway of structure.

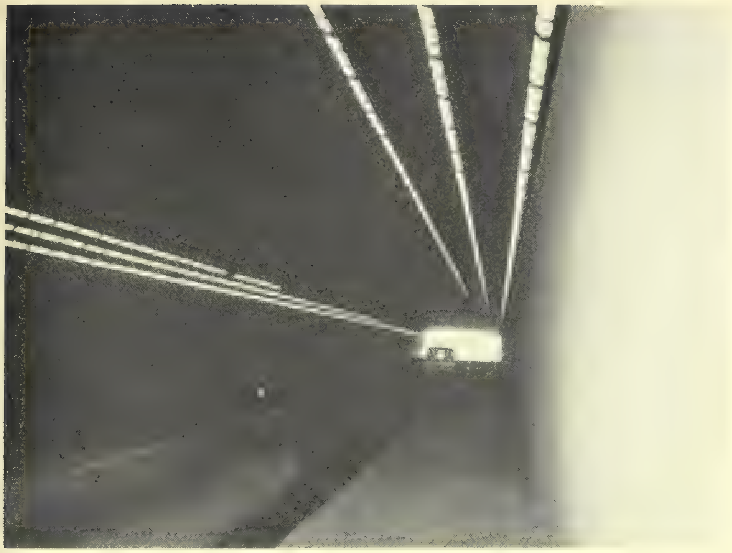

Figure 6.—Southwest International Airport Tunnel, Fort Worth, Tex.—daytime, all lights on.

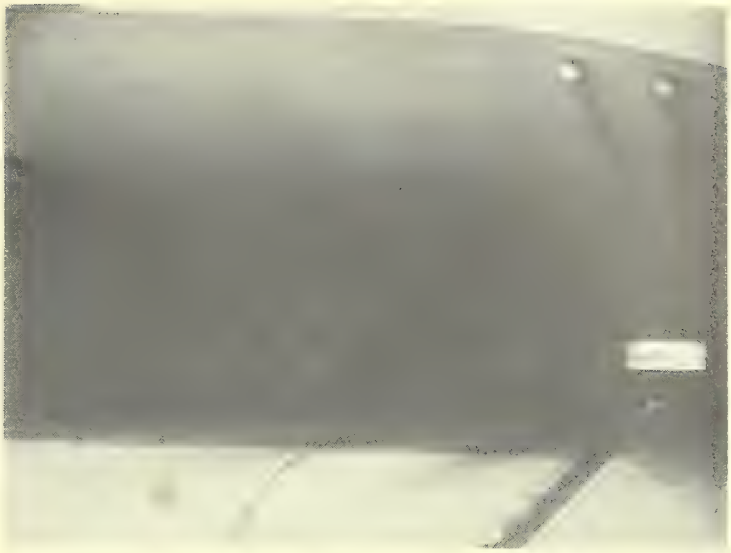

Figure 7.—Southwest International Airport Tunnel, Fort Worth, Tex.—daytime, all lights off.

structure and produces little, if any, additional visibility.

Visibility in a relatively long, straight, and level underpass can be improved by providing an opening in the ceiling about midway through the structure to permit daylight penetration, as shown in figure 5. A ceiling opening may eliminate the need for an artificial daytime lighting system.

In effect, the open ceiling provides two structures instead of one, and may satisfy the requirement for lighting during the day.

Walls and ceilings lined with a light-color matte-finish material will enhance the natural light penetration into the structure. A design with a funneled-up ceiling height at the portal will permit more effective daylight penetration. A white or light-colored pavement surface inside the structure,[3] and a black or dark-colored pavement surface outside the structure, combined with a low reflectance finish on the face of the portal, will improve the visibility without artificial lighting.

If the underpass is not on a straight and level roadway, the silhouette effect may be reduced or become nonexistent, causing relatively short structures to require daytime lighting. However, in this report, an underpass is considered to be a structure that does not require lighting during the day, and a tunnel is considered to be a structure that requires artificial lighting during the day.

Short Tunnels

A tunnel is defined as short when no more than 5 seconds are required to travel through it at the speed limit or design speed. For example, if the posted speed is 55 m.p.h., a tunnel no longer than 400 feet is considered a short tunnel.

No tunnel is long enough to allow the retina of the eye to completely adapt to nighttime light levels during the day (*4*, *5*). Unlike iris (or pupil) adaptation, which occurs in 3 or 4 seconds, complete retinal adaptation to a dark environment takes as long as an hour.

Physical design of the tunnel approach, and exit roadway may invite special lighting problems that could require extending the length of daytime entrance lighting to 6 or 7 seconds' traveltime. The same factors that are used to determine the difference between an underpass and a tunnel may also be used to determine lighting requirements that vary from standard guidelines. This is particularly true of short tunnels. Resolving that a structure is a tunnel rather than an underpass may be indefinite, and consequently, the lighting requirements may be nebulous. It is in this vague area that an in-depth evaluation is required prior to making design decisions.

Factors to be considered are length; cross-section dimensions; lining; traffic volume and speeds; location; roadway alinement on the approach, in the structure, and on the exit roadway; type of approach and exit roadway (depressed or elevated); and the direction the portal faces. A thorough evaluation of all pertinent factors should precede preliminary design.

The Southwest International Airport Tunnel on State Highway 183 in Fort Worth, Tex. (figs. 6 and 7), may have been appropriately classified as an underpass. This structure is located on a straight, level, depressed roadway with rounded back slopes that open the roadway, even at the structure portals, so as to negate the depressed features. It serves as a grade separation between Highway 183 below and the Southwest International Airport north-south runway above. As it is located within the airport, roadway lighting outside the structure is restricted to 14 fluorescent luminaires mounted 20 feet high to transition

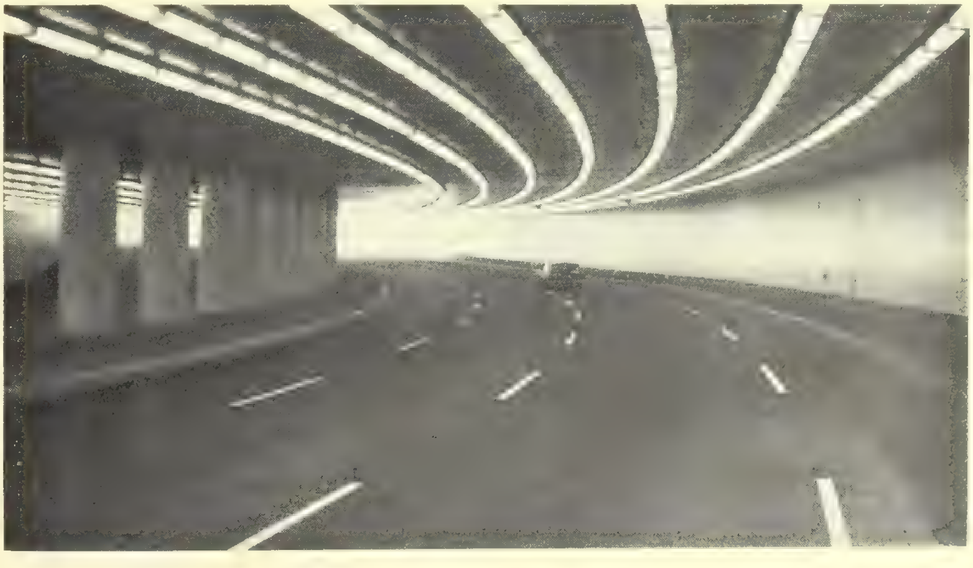

Figure 8.—Virginia-New Hampshire Avenue Tunnel, Washington, D.C.—northbound from portal, daytime.

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[3] Master's thesis, Hristaki Sofokidis (see acknowledgments).

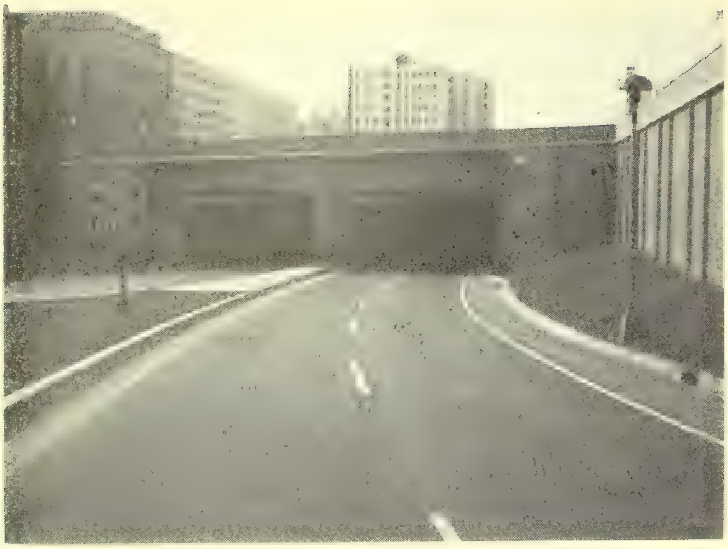


Figure 9.—E Street Tunnel, Washington, D.C.—westbound approach, daytime.

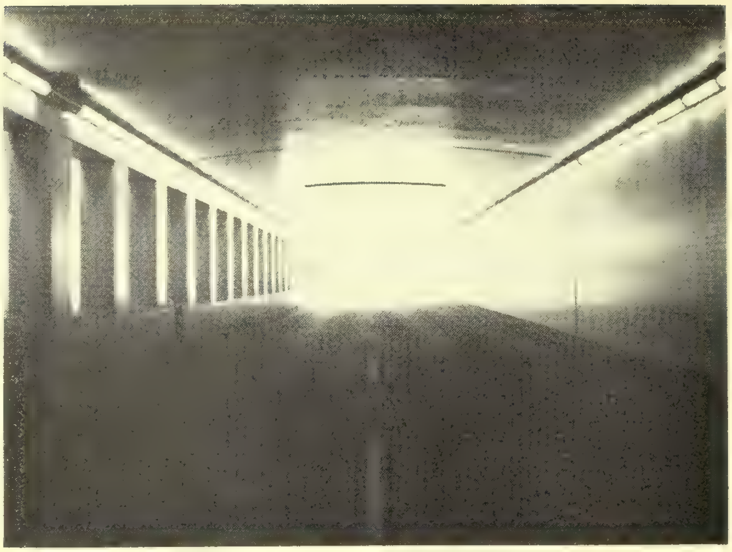


Figure 10.—E Street Tunnel, Washington, D.C.—interior near westbound exit, daytime.

from the lighted structure to the dark highway at night. The portal facades are painted gray to reduce their brightness during the day.

A relatively high level of night lighting of the type in the Southwest International Airport Tunnel exists in many of the tunnels in this country. The cost of operating lighting at higher then necessary levels is not justified for tunnel visibility and the high light levels at night are frequently the cause of hazardous conditions at the tunnel exit.

A reevaluation could classify the Southwest International Airport Tunnel as an underpass rather than a tunnel. Sufficient artificial lighting to provide enough brightness to compete with the natural daylight penetration from both ends of the structure is impractical. Very little, if any, additional visibility is provided by the 65-footcandle daytime entrance lighting. Based on observations on a bright day with the daytime artificial lighting turned off, reclassification of the structure as an underpass should be seriously considered. Because of its location, the expense of daytime lighting may not be justified.

An investigation of the lighting revealed that a 12-footcandle, nighttime illumination is

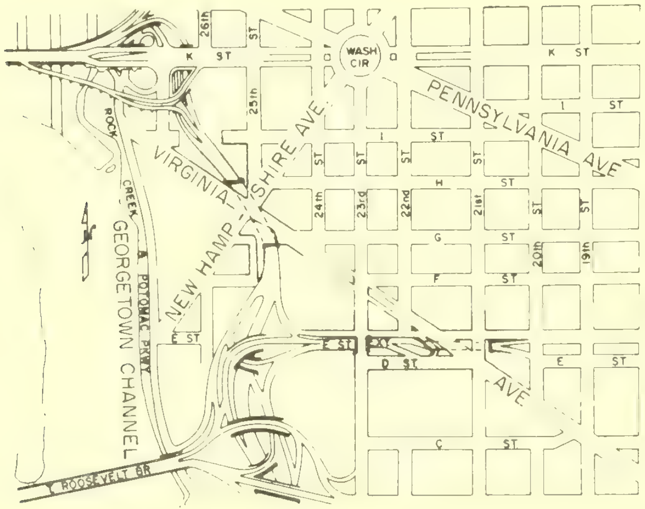
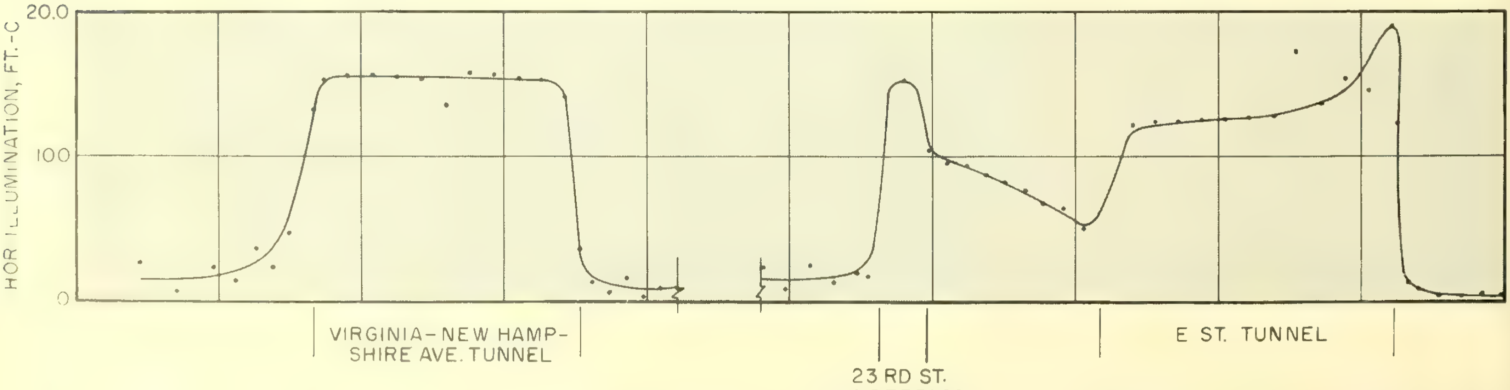


Figure 11.—Light profile at night for E Street, 23d Street, and Virginia-New Hampshire Avenue structures.



more than necessary and that a one-third reduction may still leave it in excess of the desirable level. Because the adjacent roadway lighting is restricted, the contrasting dark and light environments may require minimum lighting to avoid a hazardous condition on leaving the tunnel.

Photographs of the Virginia-New Hampshire Avenue and E Street tunnels in Washington, D.C., are shown in figures 8–10, and the physical and lighting characteristics of these structures are given in figure 11 and in tables 2 and 3. According to the light profiles in figure 11, there is an undesirable variation of roadway and tunnel (or underpass) illumination in both structures. The higher levels of night lighting in the structures tend to negate the lighting on the roadways beyond the exits. To the westbound road user of the E Street structure, this situation is critical beyond the 23d Street underpass because a major fork in the roadway necessitates a motorist's decision about 200 feet west of the underpass. Such night driving situations can be avoided by an artificial lighting environment in the underpass that is similar to the lighting on the exit roadway. Decreasing the structure lighting, as opposed to increasing the roadway lighting, is the most desirable way of correcting the condition.

Table 2.—Measured illumination in Virginia-New Hampshire Avenue Tunnel, Washington, D.C.

Tunnel station[1]	Roadway, horizontal illumination								Wall, vertical illumination		
	Line A[1] Lights on		Line B[1]			Line C[1]					
			Lights on		Lights off	Lights on		Lights off	Lights on		Lights off
	Day	Night	Day	Night	Day	Day	Night	Day	Day	Night	Day
	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.
0+00	...	...									
0+28	168	14.0	169	14.0	37.4	154	16.6	42.5	121	11.1	42.0
0+78	144	15.8	155	15.2	23.0	151	17.3	42.5	114	13.1	8.6
1+28	142	15.6	155	15.3	3.8	146	17.2	9.2	110	13.2	2.8
1+78	128	15.2	144	15.5	1.7	135	17.4	2.8	105	13.2	1.2
2+28	142	16.6	154	15.6	0.6	142	16.6	1.1	108	13.2	0.7
2+78	130	[2] 12.9	149	[2] 13.3	0.2	142	16.4	0.6	102	[2] 12.8	0.7
3+28	140	16.4	144	15.2	0.4	137	16.2	0.4	105	13.2	0.7
3+78	139	15.6	144	15.5	0.3	137	16.8	0.9	106	13.5	0.7
4+28	147	16.6	160	15.6	0.9	146	16.8	0.8	111	13.5	1.5
4+78	137	16.4	155	15.5	1.6	146	16.3	1.3	113	13.6	2.5
5+28	160	16.1	160	15.1	15.8	155	16.0	14.0	120	13.2	11.3
5+57	...	...						...			...

[1] Tunnel stations and lines are shown in figure 12.

[2] Ventilation opening in ceiling near this point.

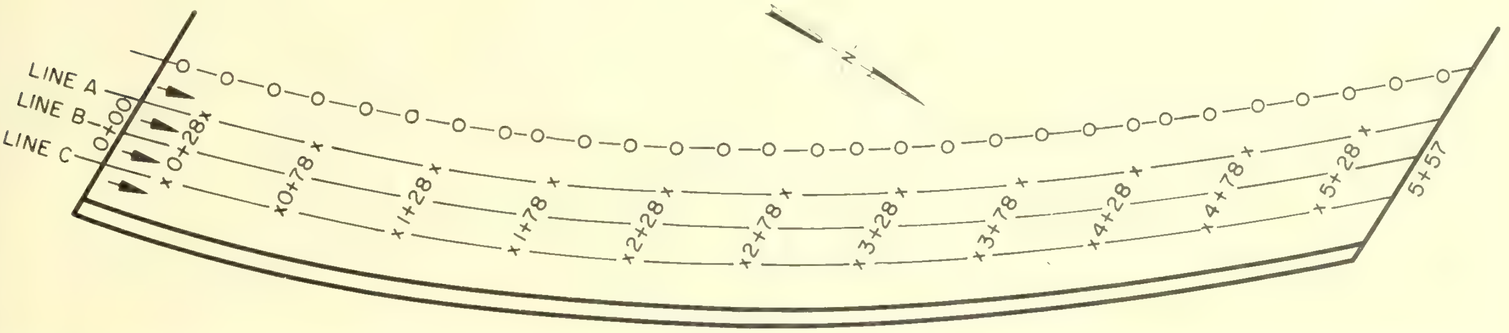


Figure 12.—Station numbers and lines used for measuring illumination in Virginia-New Hampshire Avenue Tunnel, Washington, D.C.

Table 3.—Measured illumination in E Street Tunnel, Washington, D.C.

Tunnel station[1]	Roadway, horizontal illumination			Wall, vertical illumination	
	Lights on		Lights off	Lights on	
	Day	Night	Day	Day	Night
	ft.-c.	ft.-c.	ft.-c.	ft.-c.	ft.-c.
0+00	......			...	
0+15	209		209	31.0	9.2
0+65	43	12.0	18	24.0	9.7
1+15	29	12.4	2.5	22.0	9.5
1+65	28	12.4	[2] 3.5	22.0	9.5
2+15	25	12.4	0.4	20.0	9.5
2+65	24	12.5	0.6	18.0	9.5
3+15	24	12.8	0.3	18.0	9.5
3+65	24	12.8	..	25.0	11.3
4+15	36	17.2		28.0	10.1
4+65	39	13.7		30.0	10.6
5+15	43	15.3		32.1	10.7
5+65	52	14.5		73.5	11.8
6+15	200	19.0			
6+20	.	..		..	

[1] Tunnel stations are shown in figure 13.

[2] Ventilation opening in ceiling near this point.

It is highly desirable to maintain a constant and continuous lighting environment for the road user at night on all sections of the highway including tunnels and underpasses. Illumination in structures and on adjacent roadways should vary no more than 3 to 1 and preferably no more than 2 to 1.

The daytime lighting in the Virginia-New Hampshire structure may be a good example of economically impractical lighting that provides artificial illumination of sufficient brightness to compete with the natural daylight from the ends of the structure. Visibility in the tunnel is improved very little by the artificial lighting system, which provides as much as 150 footcandles on the roadway and 100 footcandles on the walls. A comparison of conditions with and without artificial lighting showed that natural light penetration provided reasonable visibility and that the requirement for daytime lighting is questionable. The cross-section area of the tunnel enhances natural daylight penetration. Artificial daytime lighting probably would not have been considered had the structure been on a straight and level roadway; even with the existing curvature in the structure, the need for daytime lighting is questionable.

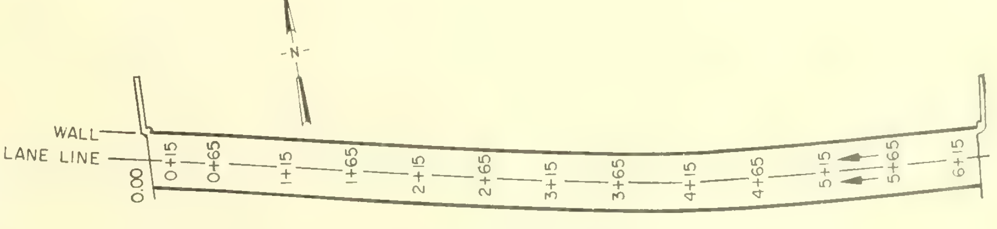


Figure 13.—Station numbers used for measuring illumination in E Street Tunnel, Washington, D.C.

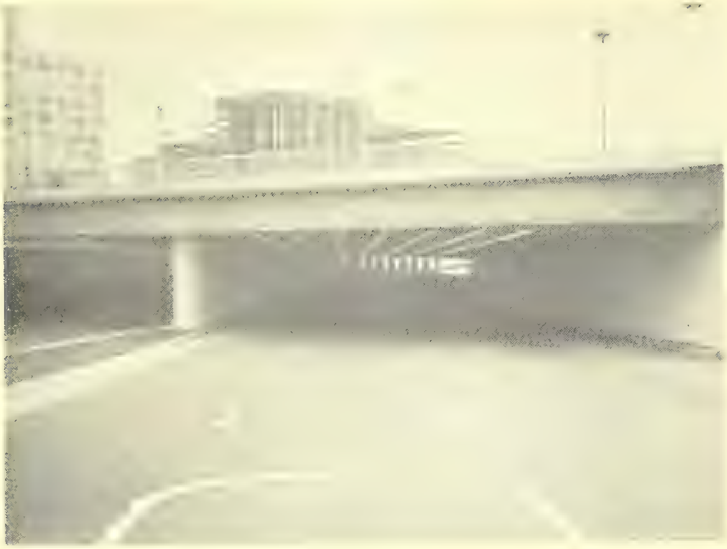


Figure 14.—George Washington Bridge Lower-Level Tunnel—near entrance, dusk.

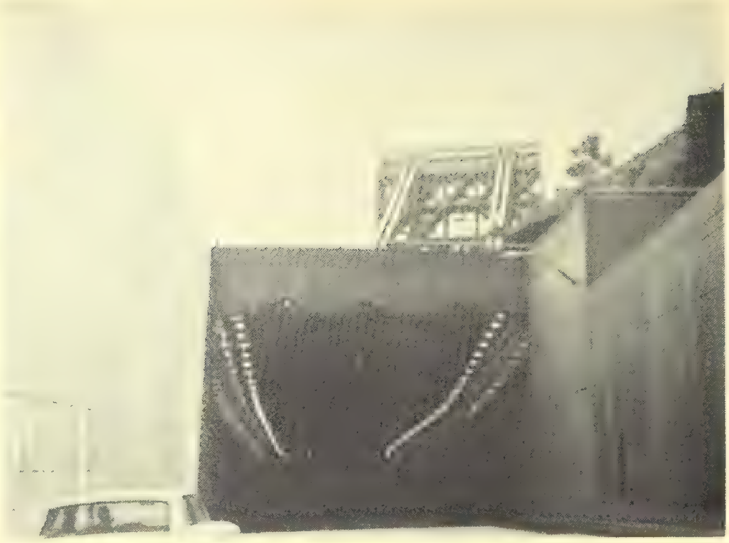


Figure 15.—George Washington Bridge Lower-Level Tunnel—approach, daytime.

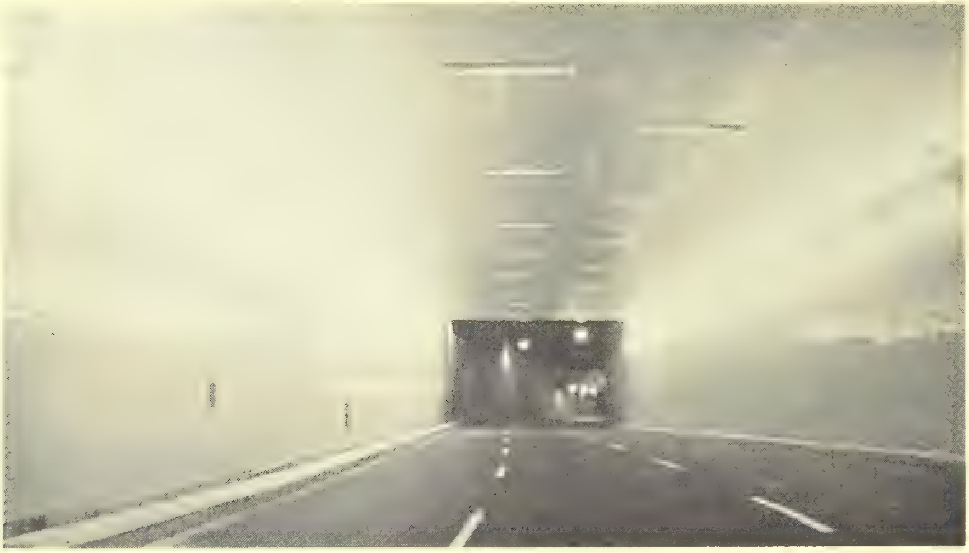


Figure 16.—George Washington Bridge Lower-Level Tunnel—near entrance, nighttime.

The George Washington Bridge Lower Level Tunnel lighting design provides a new concept that merits special attention.[4] This tunnel (figs. 14-16) uses continuous fluorescent lighting for night lighting and for the normal day zone. Supplemental daytime entrance zone lighting is accomplished with mercury luminaires. For the night and the normal day (interior) zone lighting, 6-foot, 350-milliampere slimline fluorescent lamps are wired so that two lamps are connected in series with a choke across a 1,000-volt, three-phase supply. The supply voltage can be reduced to 800 volts or raised to 1,200 volts to vary the lighting intensity. In addition, entrance (intensive) zone lighting, consisting of 400-watt clear mercury luminaires, spaced 5 feet on center for the first 150 feet, and 10 feet on center for the next 100 feet, is provided for the eastbound tunnel. These luminaires are recessed into the tunnel ceiling and are close to and directed toward the walls to produce 100 footcandles on the walls in the first zone and 65 footcandles in the second zone.

The mixing of fluorescent and mercury lamps, with the yellow tile tunnel lining, produces pleasing and effective lighting in the daytime. While the validity of transitioning the intensive zone may be questioned, the road user is provided with one of the best examples of tunnel entrance lighting in the United States.

Long Tunnels

A long tunnel is generally considered to be one that is longer than 1,000 feet, but because of physical or geometric design, some may be less than 1,000 feet in length. A long tunnel has physical, pschological, technical, and economic aspects that are different from those of short tunnels and underpasses.

The long tunnel requires two daytime lighting systems—one for the intensive zone (daytime entrance lighting) and another for the normal day zone (daytime interior lighting). Adaptation level of the road user approaching the tunnel entrance and rate of adaptation may be identical for long and short tunnels; therefore, intensive zone lighting requirements may be the same for both types of tunnels. Beyond the intensive zone is the normal day zone which is referred to as the interior. Daytime lighting of the interior is a separate consideration, and design requirements depend to a great extent on quality and type of lighting in the intensive zone. The intensive zone should allow the road user to adapt himself to the change of brightness in the tunnel. If an adaptation of 5 seconds' traveltime is satisfied by the intensive zone, normal day

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[4] Paper by Henry W. Wensen, Jr., and H. S. Lewis (see acknowledgments).

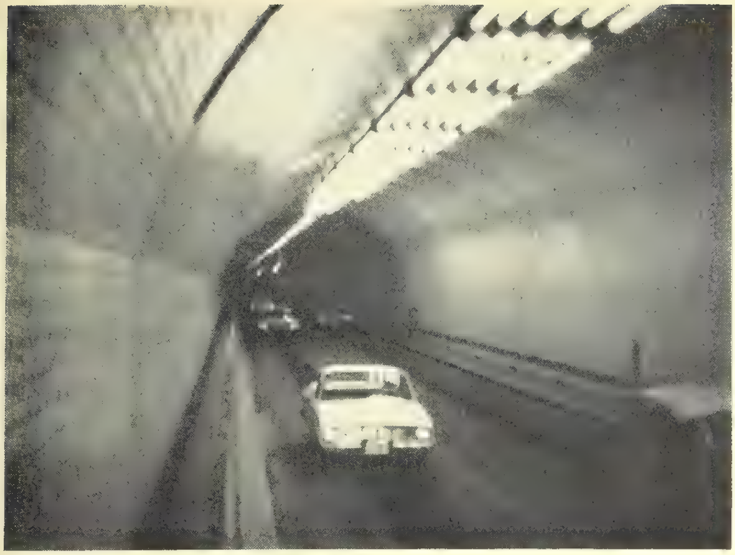

Figure 17.—Baytown Tunnel, Houston, Tex.—entrance zone, daytime.

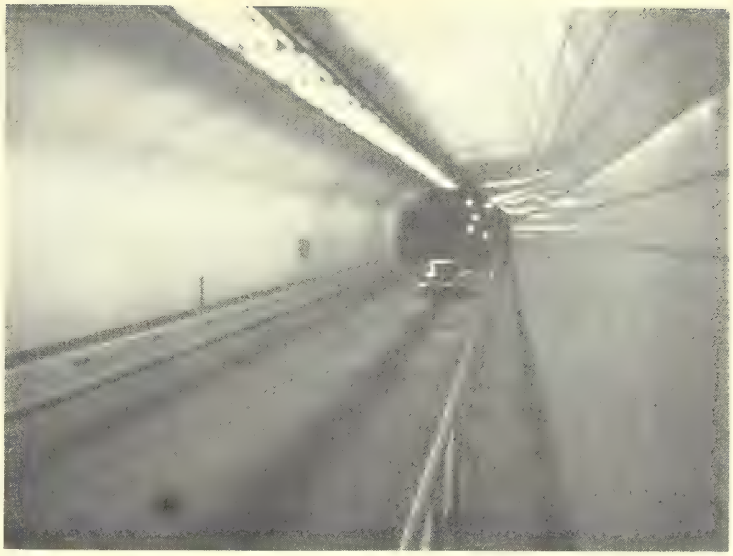

Figure 18.—Baytown Tunnel, Houston, Tex.—near exit, nighttime.

zone lighting may be designed as a separate lighting system.

As intensive zone lighting cannot be designed for complete retinal adaptation, it is necessary to have a higher level of illumination in the interior during the day than at night (*4*). Night lighting and normal day lighting for the interior are often the same not only because design problems can be simplified by one system but also because available lighting equipment may lack design flexibility. As shown in table 1, the normal day zone (and night lighting) illumination ranges from 2.5 to 40 footcandles, which are roadway values. Wall illumination was furnished for a few tunnels and ranged from 1.5 to 5 footcandles.

The primary objective in all except wide tunnels is to provide lighting on walls, and, to a lesser extent, on the ceiling. Uniform brightness is best achieved with a light-colored matte-finish surface. Although several of the older tunnels and a few of the newer ones have painted walls and ceilings, the additional cost of tile or porcelain-enamel panels may be justified on the basis of performance and maintenance.

The interiors of existing long tunnels are lighted by four basic types of lighting equipment. The first type, in some of the older tunnels, consisted of incandescent equipment that was recessed in the tunnel walls during initial construction, providing intermittent flush windows which can be easily cleaned. This lighting system is not recommended because it is inflexible and relatively less effective.

A second type, the fluorescent luminaire with internal reflector, is placed either on the walls or in the angle between the ceiling and the wall and is one way of adding new lighting to an old tunnel. This type of luminaire has been installed in some recently completed tunnels. It does offer flexibility for future new lighting, but must be watertight, bugtight, and dusttight for efficient lighting and maintenance.

A third type, the fluorescent luminaire with reflector-deflector, was developed by the California Division of Highways.[5] This luminaire is similar to the internal reflector, except it has reflector louvers to direct the light to the walls and ceiling, and delivers only indirect light to the roadway.

The fourth type consists of a fluorescent lamp inside a protecting Pyrex glass tube, mounted a few inches from the tunnel wall or ceiling. Compared with the others, this type of luminaire is easier to make watertight and is being used in several newly constructed tunnels.

Continuous fluorescent equipment is the best interior lighting for preventing undesirable flicker. It is difficult, and often impossible, to provide both sufficient brightness for intensive zone lighting and reduced brightness for interior or night lighting using the same equipment without additional luminaires, even though variation in voltage or current is possible in some lighting systems. For this reason, it seems undesirable to restrict the design choice to the same type of equipment for both intensive and interior zone requirements. Where lower lighting levels are required, as for interior and night lighting, fluorescent lighting is desirable and is the type most commonly used.

The most difficult section of the vehicular tunnel to illuminate is the entrance section. The lighting of this section is one of the few problems in illuminating engineering in which both day and night levels of lighting are common considerations. This critical lighting problem demands maximum analysis and evaluation, and an understanding of several professional fields. It is evident that entrance lighting has been a problem in many tunnels in this country, and it is equally evident that the problem has not been properly solved. The underdesign of the daytime entrance lighting occurs as often as the overdesign of the night lighting.

In some daytime entrance lighting designs, illumination of the roadway has been given major importance when roadway brightness should have been incidental to wall brightness. The most common occurrence, however, is simply insufficient lighting. Responses from interviews with those responsible for the designs ranged from a complete misunderstanding of the entrance lighting problem to an acute awareness of it. Weakness in tunnel entrance lighting practices stemmed from two primary practices: (1) Attempting to use the same luminaire or type of luminaire to satisfy three lighting conditions, and (2) designing for eye adaptation within the tunnel without recognizing the fact that difficulty is experienced by the driver not when he has entered the tunnel, but when he is still outside of it.

While outside the tunnel, the driver is adapted almost to full daylight brightness and is trying to look into the much darker tunnel interior. The Texas Highway Department's evaluation of the daytime entrance lighting in the Baytown Tunnel (figs. 17 and 18) under the Houston Ship Channel in Harris County, Tex., is typical of prevailing conditions in many other tunnels—satisfactory on a cloudy day, and unsatisfactory on a bright day. Better designed daytime entrance lighting is needed for this tunnel. Night lighting is provided by two rows of luminaires that produce about 8 footcandles on the roadway in the tunnel. Roadway illumination outside the tunnel is about 1 footcandle. When one row of the luminaires was turned off, the visibility in the tunnel was not impaired, but the transition from the tunnel interior to the outside roadway was greatly improved. This same condition was observed in evaluations of many tunnels in which it is indicated that the night lighting in a tunnel should be kept to a minimum and should always be considered in relation to the lighting on the roadway outside the tunnel. This does not imply that the roadway lighting should be increased, but that the tunnel lighting should be reduced so that the ratio of roadway lighting inside the tunnel to the roadway lighting outside the tunnel should not exceed 3 to 1 and preferably 2 to 1.

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[5] Reported by Harold Skootsky and John R. Brass (see acknowledgments).

Visibility was judged as satisfactory when vehicles were clearly seen as silhouettes against the tunnel walls or ceiling. Very low, but uniform, wall and ceiling brightness will provide this visibility. Tiling or paneling of walls and ceiling enhance lighting uniformity and decrease the illumination requirements.

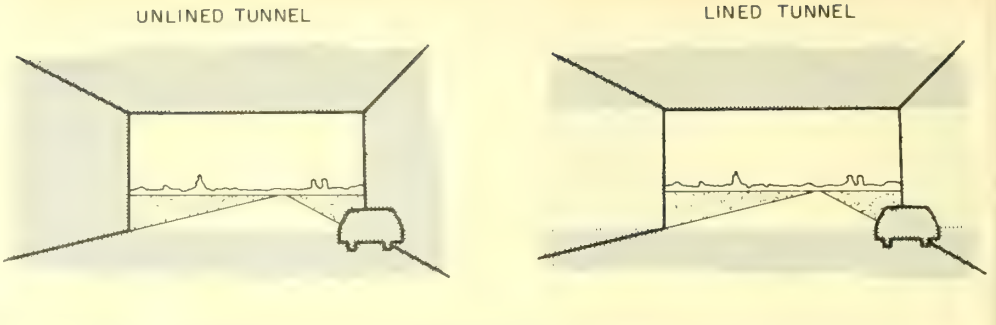


Figure 19.—Effect of natural light penetration on walls at the tunnel exit.

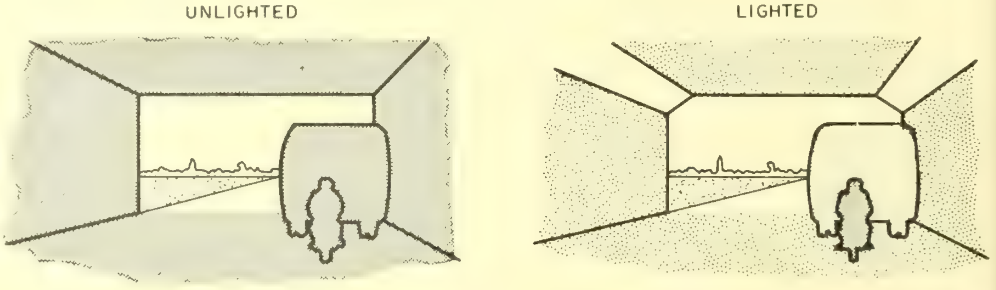


Figure 20.—Effect of normal day zone lighting at the tunnel exit.

Conclusions

Daytime entrance lighting

During daytime operation, tunnel entrances should have a supplemental lighting system to satisfy the visual requirements and adaptation of the road user as he approaches the tunnel, as he enters the tunnel, and after he enters it. Adaptation over the ranges experienced in entering a tunnel occurs very quickly; therefore, the basic requirement of the entrance lighting is to provide the range of adaptation and not the rate.

Many types and combinations of luminaires have been used to meet these demands because the prevailing characteristics are rarely identical for any two tunnels. Tunnel inspection in the United States showed that there are few good examples of lighting, several examples of fair lighting, and many examples of poor lighting. None was completely in accord with the Illuminating Engineering Society's 1957 report entitled *Lighting Traffic Tunnels and Underpasses.* However, this fact alone would not necessarily deem them unsatisfactory, as this report is outdated and is now being revised.

Using equipment that is available, innovation and good engineering can provide a product that will satisfy the need for three lighting systems: (1) Daytime entrance; (2) normal day zone; and (3) night.

If one piece of hardware will not accomplish all three needs, then more than one should be used. Therein lies the solution to many of the existing problems of tunnel lighting. Economic considerations should prevail only after the minimum functional requirements are achieved.

Although the majority of entrance lighting has been provided with additional fluorescent luminaires similar to those used for normal day zone and night lighting, the high brightness desired may best be accomplished with other types of luminaires. Economy, which is always an engineering factor, may be achieved by investing more in the entrance lighting and less in the night lighting.

Exit lighting

During the day, the tunnel exit appears as a *light hole* to the road user. Obstacles on the roadway will stand out as a silhouette against the exit and thus be clearly visible. This silhouette visibility at the exit can be further enhanced by the additional natural light penetration into the tunnel produced by tile- or panel-lined walls at the exit, as shown in figure 19. Thus, no visibility problem exists as long as the tunnel exit is clear.

When the exit is not clear, there may be a different visibility requirement. When a small object is following a larger one, as shown in figure 20, the smaller object would not be obvious and, depending on its color, may not be visible. This situation is improved by the normal day zone lighting, and is sufficient justification to extend the normal day zone lighting to the exit portal. Supplemental exit lighting is rarely required at the tunnel exit.

The most common difficulty at the tunnel exit occurs at night, when the roadway outside the tunnel has no lighting or a lower level of illumination than inside the tunnel, and the exit appears as a *dark hole* to the road user. Excessive lighting inside the tunnel is the common cause of this situation. Many tunnels have the same system for night lighting as for normal day lighting. Inasmuch as the normal day lighting system must provide substantially more brightness than is required for the night lighting, a single lighting system, with no provision for varying the amount of illumination, cannot satisfy both requirements. At night, lighting in excess of the optimum is detrimental and may create a hazardous condition at the exit. The conditions for road user visibility are not met if the normal day lighting in the tunnel remains the same at night. The California State Highway Department designed a luminaire with control circuits to change lamp operation for daytime, and nighttime conditions. Two 1,400-ma. lamps and one 600-ma. lamp are for daytime operation and one 60-ma. lamp is used at night. Actually, the lamp that is operated at 600 ma. during the day is operated at 60 ma. at night. This luminaire satisfies the day and night lighting requirements for the tunnel interior but does not furnish sufficient illumination for most daytime entrance lighting needs.

Flicker

Discontinuous lighting in long tunnels results in nonuniform brightness of the interior and produces a recurrent flicker which discomforts the road user. It also reduces visibility by producing phychological harassment in the form of moving shadows within the driver's vehicle, and troublesome reflections that appear to crawl over other vehicles and, to some extent, camouflage them. This phenomena, which is the result of periodic brightness changes, is dependent on the repetition frequency. When the frequency is increased to 50 to 80 cycles per second, called the flicker-fusion frequency, the periodic character of the lighting can no longer be distinguished.

In the United States, flicker causes few problems because most tunnel interiors are illuminated by continuous lines of lighting. Fluorescent luminaires installed in continuous rows eliminate flicker. Even though the end plates, fittings, etc., interrupt the continuous lighting, they are not of sufficient length to cause objectionable flicker. Continuous lighting must be installed parallel to the roadway; the mounting of a linear source laterally across the ceiling of a tunnel can be glaring and cause flicker.

Continuous fluorescent tunnel lighting systems are recommended for night lighting and interior day lighting.

Maintenance

Reliability of tunnel lighting is vital because it is used continuously, 24 hours a day. The lighting depends on many factors that should

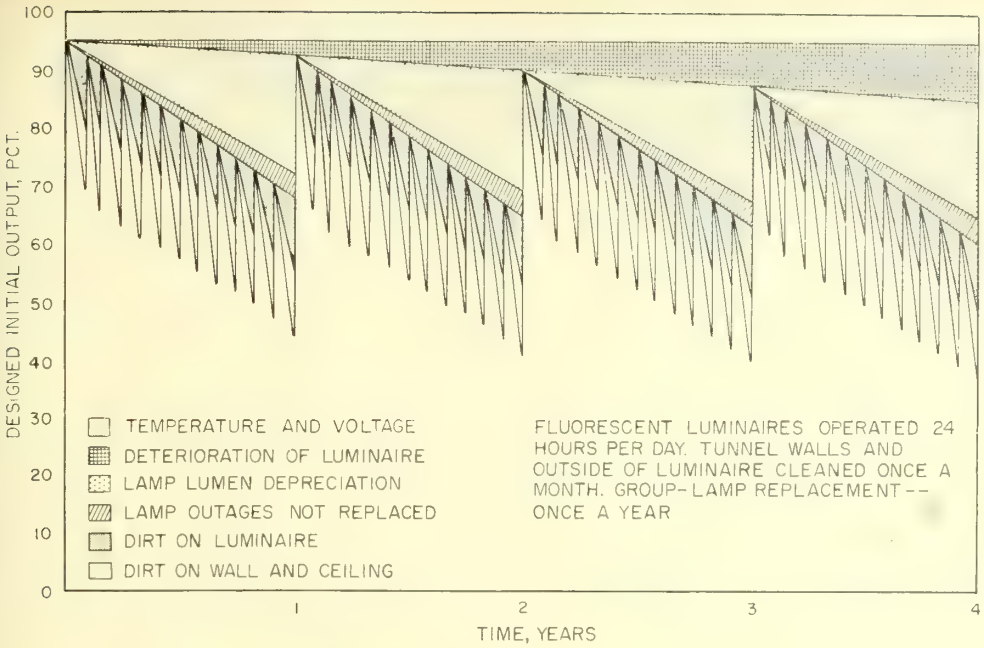


Figure 21.—Effect of a preventive maintenance program—six causes of light loss for a tunnel interior lighting system.

be recognized and accounted for at the design stage. Maintenance of the lighting is a most important factor, and the proposed maintenance program must be reflected in the initial design or realistic results cannot be forecast (fig. 21). Poorly maintained tunnels discredit the responsible agency, and present ineffective lighting with its associated hazards.

The maintenance required depends on the location, type, and volume of traffic; type and capacity of the ventilation system; tunnel cross section and shape; ceiling and wall finish; operating speeds in the tunnel; type and location of luminaires; grades and alinement within the tunnel; and the electrical system and supply. Even momentary failure of the lighting is dangerous, and adequate standby equipment and supplies are necessary. There is much more equipment in a tunnel than in a corresponding length of ordinary roadway, and tunnel maintenance is a large undertaking.

A tunnel must have sufficiently high initial illumination to compensate for the many factors that will reduce it. The most important factor, with the exception of the luminaries is the surface treatment of the walls and ceiling. Reducing the reflection factor of these surfaces may be more detrimental to lighting effectiveness than any other single factor. Vehicular tunnels must be finished with an interior surface that will not deteriorate as time progresses and as chemicals attack it, that will not readily soil, and that can be easily cleaned. These attributes are characteristics of a light-colored matte-finish tile or porcelain-enamel panel with an initial reflectance of 70 percent or higher. The temptation to use a less costly finish should be resisted because the cost of cleaning and repainting, plus the loss of reflectance, will result in much higher maintenance costs and require more initial illumination.

Luminaires should be sealed to prevent the entry of dust and water when the tunnel is being cleaned by a high-pressure spray. They should be designed to permit quick and easy internal cleaning. The materials used should be resistant to alkaline deposits, to concentrated exhaust fumes, and especially to harsh cleaning solutions that must be used to thoroughly clean the tunnel walls and ceiling.

More frequent tunnel cleaning is required when traffic volumes are high and there is a large percentage of truck traffic. Cleaning also is required more often when the vehicular speeds are slow and ascending grades exist within the tunnel, as exhaust fume deposits are a major cause of tunnel dirt.

Because exhaust fumes are a major cause of tunnel dirt, design of the ventilation system is a factor to be considered in the design of the lighting as well as in maintenance programs.

Regardless of the difficulties, lamps should be replaced on a group replacement program, which not only will help preserve the light output at the desired level, but also provide for a balanced maintenance workload. The magnitude of relamping will generally require group replacement by sections. To replace lamps on a burnout basis is usually false economy from the standpoint of lighting output and equipment maintenance costs. Fluorescent ballasts and lamp holders can be damaged by very old lamps that develop rectification and sputtering. Even though some lamps will burn for years, eventually their light output will be reduced to a point beyond which it is economical to keep them in service. Design for a specific level of illumination is impossible when lamps are replaced on a burnout basis only.

The inside of the luminaire will require cleaning more often than the frequency of lamp replacement, but maintenance schedules still should prescribe cleaning each time a lamp is replaced.

Some tunnels need cleaning weekly, whereas others may not require cleaning for several weeks. Locations in northern sections of the country, where freezing conditions may exist continuously for several weeks or months, make cleaning operations difficult and frequently impractical. These are factors that must be recognized, evaluated, and accounted for in the design of the lighting system. Designs

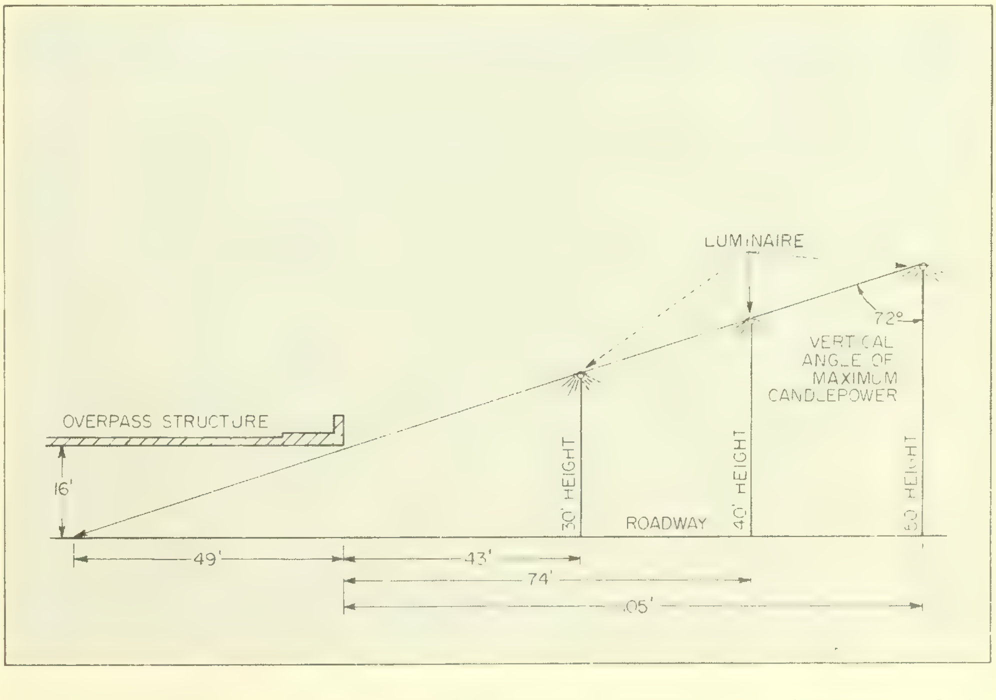


Figure 22.—Location of roadway lighting luminaires adjacent to underpass portals for maximum penetration.

that do not include provisions for maintenance are incomplete and ineffective.

Entrance roadways

Nighttime roadway lighting adjacent to the tunnel entrance is not required for visibility but may be provided without detrimental effect. The criteria for roadway lighting should be the same as though no tunnel existed. When the roadway is continuously lighted, no change should be required because of the tunnel.

Special designs have been successfully used, and are recommended when practical, to reduce the daytime brightness of areas outside the portal and enhance the visibility of the road user. A low reflectance finish on the face of the portal, dark pavement surface, and heavy tree foliage shading the roadway and portal area will reduce the adaptation level of the driver.

Light transition louvers have been used on the approach roadway to reduce the brightness ratio between the tunnel interior and the exterior environment (*12*). A few installations have been judged favorably, and many illuminating engineers believe more use should be made of louvers, which shade the roadway ahead of the entrance and provide a light transition between the bright daylight of the exterior and the darker tunnel entrance.

Exit roadways

There is no need for special treatment at the exit roadway during the day. With properly designed night tunnel lighting and roadway lighting adjacent to the tunnel exit, there should be no problem at night. However, when the lighting in the tunnel exceeds the requirements for visibility, the additional expense of designing higher-than-necessary roadway illumination to provide transition from the excessively bright tunnel interior to the relatively dark exterior roadway should not be condoned. The night lighting should be much less than the normal day lighting to avoid creating potentially hazardous conditions and unnecessary expense, yet provide the road user with optimum visibility.

Guide Values for Lighting

Underpasses

Underpasses, up to 75 feet in length, may be adequately lighted from roadway luminaires outside the underpass. Lighting poles should be located near each portal, as shown in figure 22, so that the maximum amount of light can penetrate the underpass. When supplemental underpass lighting is required, the roadway lighting in the underpass should be the same as that on the approach and exit roadway. In urban areas, additional illumination may be required for pedestrians and for the policing of underpasses. For these applications, the underpass lighting should be no more than three times the lighting on the roadway outside the underpass.

Lighting is not necessary in an underpass during the day, except for pedestrians and policing, and then only in long underpasses.

Short tunnels

Short tunnels require two lighting systems, one for daytime and one for nighttime.

Daytime entrance lighting.—The entire length of the tunnel should be designed for sufficient wall and ceiling brightness to silhouette an obstacle for the road user outside the tunnel. The required lighting will vary with prevailing conditions, but should generally average between 30 and 60 footcandles maintained on the walls. All values of wall and ceiling illumination are based on a reflectance factor of at least 70 percent for wall surfaces. When reflectance is less than 70 percent, designed illumination should be increased to compensate for the lower reflectance. (See table 4.)

Table 4.—Guidelines for maintained illumination on wall

Lighting system	Underpass	Short tunnel	Long tunnel
	ft.-c	*ft.-c*	*ft.-c*
Day entrance	--	30–60	30–60
Day interior			5–10
Night	(1)	0. 7–2. 0	0. 7–2. 0

[1] Should be same as approach roadway lighting.

When more than one row of fluorescent luminaires is required to light each wall, other types of luminaries to provide the additional illumination should be considered.

Night lighting.—Lighting at night must be designed to avoid flicker and glare, and illumination should be the same throughout the entire length of the tunnel. The ratio of the roadway lighting inside the tunnel to that of the roadway outside the tunnel should not be greater than 3:1 and preferably should not exceed 2:1. Wall and ceiling illumination in tunnels during the night should be maintained between 0.7 and 2.0 footcandles. Fluorescent luminaires are recommended for night lighting to provide continuous, uniform, low brightness lighting on the walls and ceiling.

Long tunnels

Three lighting systems are required for long tunnels, one for daytime entrance, one for normal day zone (interior), and one for night.

Daytime entrance lighting.—Requirements for daytime entrance lighting in long tunnels are identical to those for short tunnels. The length of daytime entrance lighting should provide for at least 5 seconds' traveltime in the tunnel, or provide a safe stopping distance. An additional length of intensive zone lighting may be required in a straight and level tunnel to provide sufficient background brightness to silhouette an obstacle in the tunnel. The effectiveness and economy of all types of luminaires should be considered, evaluated, and compared before selecting the type of luminaire to perform the daytime lighting task in the tunnel entrance.

Normal day zone (interior) lighting.—Interior lighting must be free from flicker and glare. A high-reflectance tunnel lining is recommended to produce adequate wall and ceiling brightness in tunnels during the day and night. The interior lighting should maintain between 5 and 10 footcandles on the walls. Fluorescent luminaires extended to the tunnel exit are recommended to provide interior lighting.

Night Lighting.—The night lighting requirements for long tunnels are identical to those for short tunnels.

Tunnel lining

The interior walls and ceiling of the tunnel are essential adjuncts of the lighting system. Wall and ceiling brightness and uniformity depend on the reflectance quality of the surfaces. The surfaces should be of a material that will not deteriorate with age and chemical attack, will not readily soil, and can be easily cleaned. Although special paints are available for use on tunnel interiors, a more permanent light color matte (nonspecular) finish surface, with a reflectance of 70 percent or greater, is recommended. When a surface material has less than 70 percent reflectance, the designed lighting should be increased to provide equivalent wall and ceiling brightness.

The roadway surface even though it is more subject to dirt and discoloration can affect the tunnel lighting. Consideration of lighting effectiveness would indicate preference for a light color pavement. An asphalt roadway surfacing would be desirable for decreasing noise, however, maintenance and economical considerations would probably be the deciding factors in the choice of roadway material.

Wide tunnels

In lighting tunnels up to about 50 feet wide, wall brightness is the principal objective; in wider tunnels, roadway lighting may become equal in importance. The objective of providing background brightness for silhouette vision may not be completely achieved in a wide tunnel from wall brightness alone. In these tunnels it may be impractical to provide sufficient background brightness; therefore, entrance lighting, preferably by ceiling mounted luminaires, must also provide direct discernment or object glint for the approaching road user.

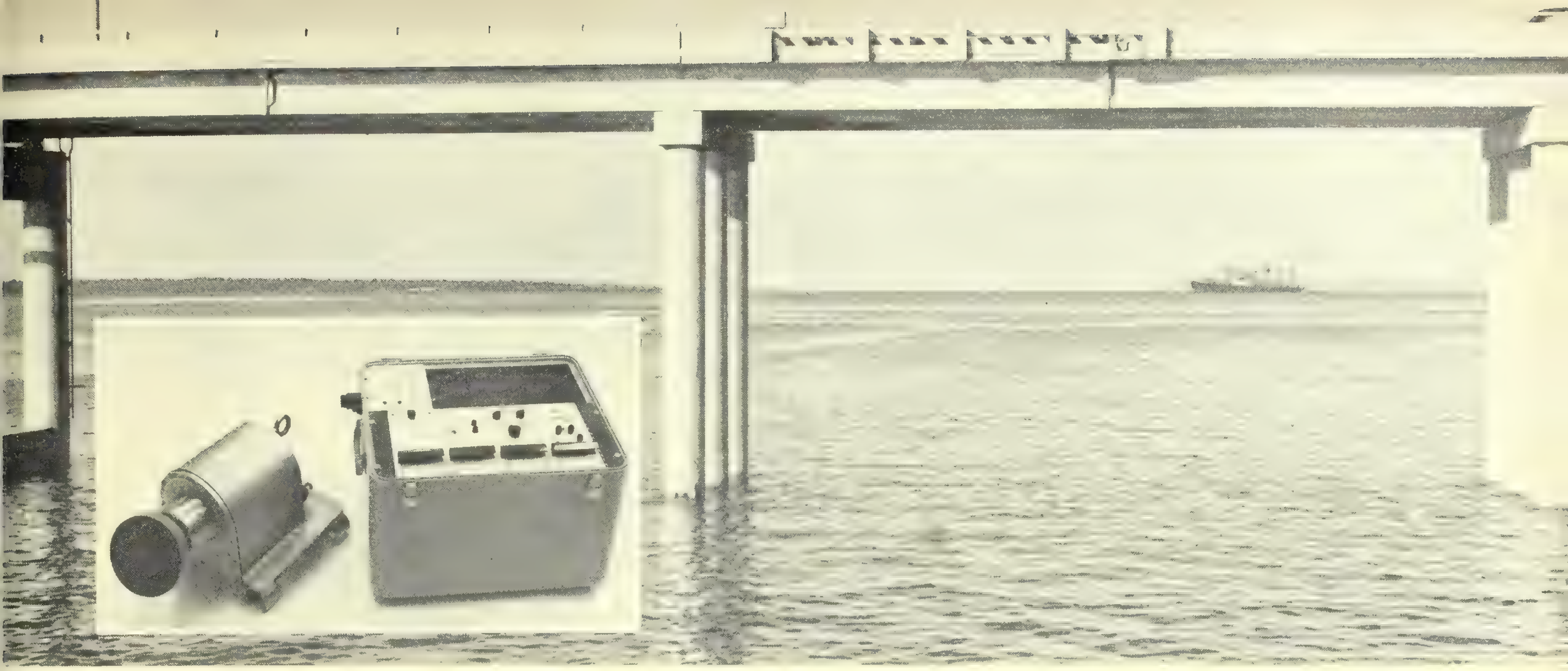
Some roadways, because of the specular nature of their surfaces and their reflectance at the grazing angles of incidence, *mirror* the wall and ceiling brightness. This *mirror* characteristic may reduce the amount of overhead illumination required.

The levels of illumination in a wide tunnel should be similar to those in any other tunnel. Luminaires providing wall and ceiling brightness in tunnels up to about 50 feet wide usually produce adequate roadway brightness. In wider tunnels, wall lighting may leave a dark center in the roadway—a condition that may be alleviated by additional lighting.

Maintenance

Tunnel maintenance cannot be allowed to *just happen*—it must be planned. To assure satisfactory lighting results, the designer must have knowledge of the proposed maintenance program. Physical maintenance is the only way to continue the effectiveness of any light-

(*Continued on p. 105*)



Youngs Bay Bridge, Astoria, Oregon, site at which newly developed scour meter was tested.

Ultrasonic Instrument for Determining Local Scour at Bridge Piers

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by MURRAY L. CORRY,[1]
Highway Engineer, Office of Engineering
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Research Engineer, Automation Industries, Inc.

The ultrasonic scour meter described in this article was developed to determine the magnitude of local scour around bridge piers. The instrument automatically obtains sufficient depth data to map the stream bottom around the upstream face of a pier. Although the instrument was designed to fill a research need, a simplified version could also be adapted to maintenance and inspection operations.

Introduction

IN 1894, Engels published the results of a model study in which he investigated local scour around bridge piers and the effectiveness of riprap in attenuating such channel degradation. Since that time, numerous other model studies have been undertaken to define the mechanics of this localized scour process, resulting in a large body of data that adequately describes the local scour phenomenon in scale models and, with the application of model-prototype similitude relations, could be used to predict local scour in a real environment. To determine these model-prototype similitude relationships, scour and related hydraulic and geometric data must be obtained around full scale structures for comparison with the model results.

The instrument described in this article was designed to obtain the local scour portion of these data. It is an electroacoustical device for determining, as a function of time, the changing streambed elevation around bridge piers. The device, called an *ultrasonic scour meter*, was developed in a research program sponsored by the Office of Research and Development, Bureau of Public Roads.

Electroacoustical Systems

Usually, any electroacoustical system designed for underwater operation requires the following elements:

- A signal generator that sends an electrical pulse to a transmitter.
- A signal transmitter (transducer), located in the water, to convert the electrical pulse into a mechanical vibration, resulting in the propagation of a pressure wave through the water.
- The media (water) in which the pressure wave is transmitted and the reflected echo retransmitted.
- A receiver (transducer), also located in the water, to detect the reflected pressure wave and convert the received mechanical energy to an electrical pulse. Often, one transducer serves as both a receiver and transmitter.
- An electronic amplifier to upgrade the pulse to a usable amplitude.
- A computer that measures the time lapse between transmission and reception and converts this information into distance between transmitter and reflector.

The basic configuration of the ultrasonic scour meter follows this general design concept (fig. 1). It consists of a transducer that serves both as a transmitter and receiver, and an electronic unit that generates the signal or pulse, amplifies the return pulse, and computes a voltage proportional to the distance between the transmitter (transducer) and the stream bottom. The unique feature of the

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[1] During the research program, Mr. Corry was in the Office of Research and Development.

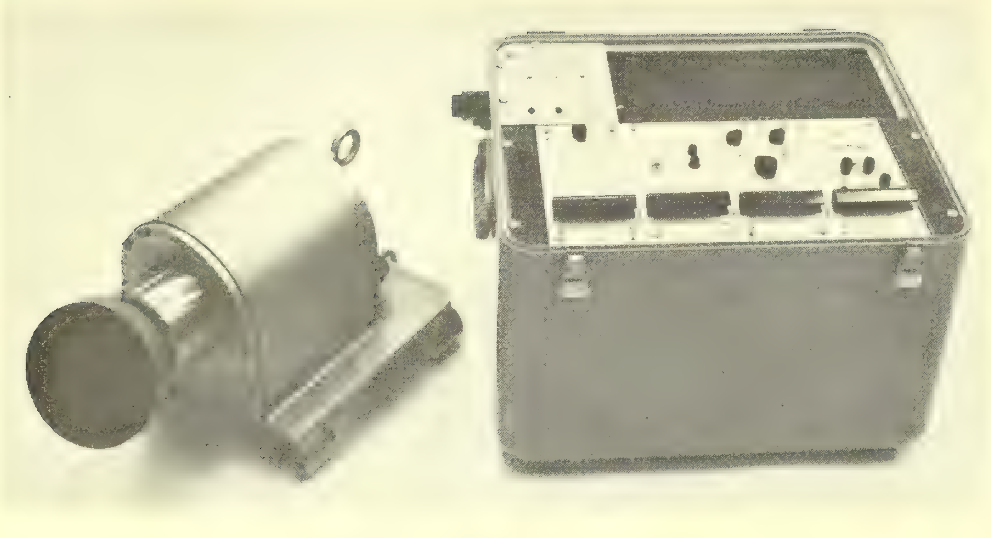

Figure 1.—Ultrasonic scour meter.

instrument is its ability to automatically obtain sufficient data to map the stream bottom near the upstream face of a bridge pier. This multipoint depth data is acquired by a mechnical unit that moves the transducer through a preset positioning sequence similar to that shown in figure 2.

Initially, a depth reading is obtained while the transducer is positioned straight down (0° vertical and minus 120° horizontal). Next, a vertical positioning gear moves the transducer to the 4° vertical position where motion is delayed for 1 second while a depth indication is obtained. A horizontal positioning gear then moves the transducer to the −90° horizontal position where motion is again delayed 1 second to obtain a depth reading. The sequence is continued through the −60°, −30°, 0°, +30°, +60°, +90°, and +120° horizontal positions, after which the transducer is returned to the original −120° position. Immediately upon return of the horizontal mechanism to the −120° position, the vertical drive moves the vertical gear to the 8° position. The horizontal sequence is then repeated, and the vertical mechanism moves the transducer to the 18° position. This sequencing continues through the vertical angles of 28°, 43°, and 61°. After completing the horizontal sequence at the 61° vertical position, the vertical drive returns the vertical gear to the 0° position in preparation for the next sequence.

In operation, the mechanical unit is positioned on the bridge pier below the water surface and is connected to the electronic unit on the bridge by waterproof cables.

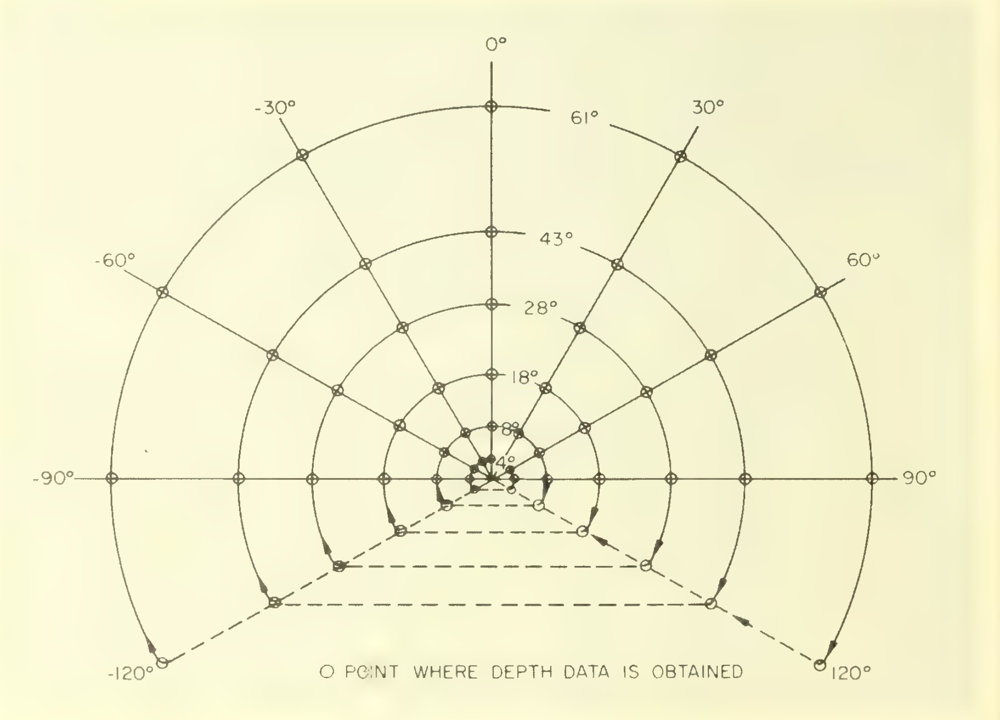

Figure 2.—Bottom mapping sequence.

Field Test

The scour meter was tested under field conditions at a highway bridge over Youngs Bay, Oregon, in December 1966. Because of the tidal action, this site had two flood, or scour, periods daily.

The instrument was positioned normal to the bridge alinement, and guidelines were strung from the instrumented pier to the piers on each side and to a railroad trestle approximately 150 feet away (fig. 3). Distance from the instrumented pier was indicated on each line. Check measurements were obtained by lead-line soundings at the pier and from a boat following the guidelines. An ultrasonic device with an accuracy comparable to that expected from the test instrument was used as the check instrument in the boat. Depth measurements were obtained by the two instruments at approximately the same time; that is, the test instrument mapped the scour hole while the ultrasonic device on the boat obtained a continuous depth record along the 0° and ±90° sounding ranges.

Both instruments recorded the shortest distance from the transducers to the streambed within the transmitted energy beams. Therefore, the depths recorded were functions of the transducer positions relative to the channel bottom. Accordingly, with an upward sloping channel bed, the test instrument should have always recorded a greater depth than that shown by the check instrument. The test results proved this to be true for most of the measurements obtained within the limits of the scour hole (angles less than 45°); but outside the limits of the scour hole (angles more than 45°), the reverse situation was encountered—the check instrument recorded the greater depth. (See figs. 4 and 5.) Outside the scoured area the discrepancies were probably due to the large angle between the transducer face of the test instrument and the streambed, which could have resulted in a high energy loss and a corresponding reduction in accuracy.

Although the tests were more qualitative than quantitative, they did indicate the feasibility of using such a device to map scour-hole areas. However, because of the difficulties encountered with the larger vertical angles, care should be exercised in interpreting the data for vertical angles larger than 45°.

The mapping capability of the test instrument is illustrated in figure 6. The contour map shown was derived from data obtained during one test sequence.

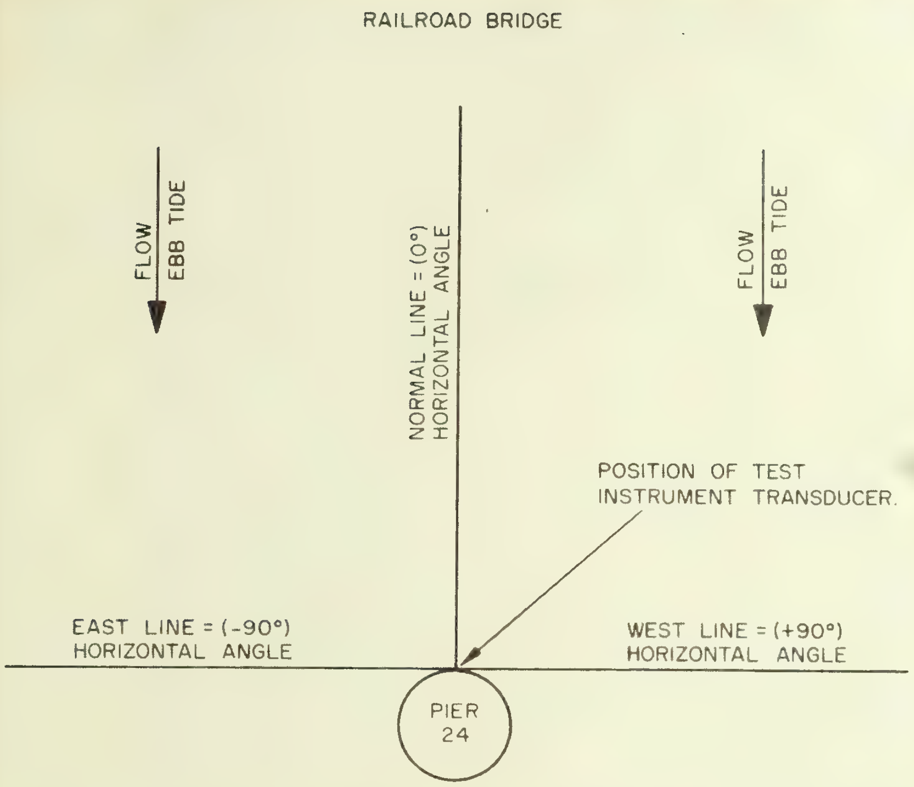

Figure 3.—Plan view of sounding lines at test location.

Instrument Development

The development program was divided into two phases—a feasibility-study phase and a design, construction, and testing phase. Hydraulic mechanical, electronic, and transducer problems and capabilities were investigated during the feasibility study.

Feasibility study

Results of the hydraulic portion of the feasibility study indicated that no detrimental effects should be experienced from turbulence, temperature gradients, air bubbles, and sediment conditions usually encountered around piers. However, swift streams may carry sediment particles of sufficient size or concentration to make detection of the bottom difficult. Measurements of ultrasonic-signal attenuation due to suspended sediment indicated that concentrations larger than 20,000 p.p.m. will greatly reduce the effective range of any pulse-echo instrument. Measured two-way attenuation figures for a 200-kilo Hertz (kHz) ultrasonic beam, using bentonite as the wash load, were 0.136, 0.336, and 1.55 decibels per foot at 10,000, 20,000, and 30,000 p.p.m. concentrations, respectively. These correspond to power losses of 3%, 8%, and 30% per foot.

From theoretical calculations for the transducer design a computer program was devised to plot the transducer directivity functions. Transducer diameters from 3 to 9 inches were plotted using a 200 kHz driving frequency. Subsequent pattern measurements, using a large tank and three transducers 2, 7, and 10 inches in diameter, indicated good agreement with the calculated values. Figure 7 is a plot of the measured directivity of the 7-inch unit.

Instrument design

Instrument design was controlled by the results of the feasibility study and by the data requirements. The design specifications required: (1) that the instrument consist of an electronic package containing transmitter and receiver modules and a mechanical unit containing the transducer and transducer positioning mechanism; (2) that the electronic package be transistorized and designed for battery operation; (3) that the electronic design provide all necessary functions for a pulse-echo system capable of ranging in increments to a distance of 150 feet with an absolute accuracy of ±0.50 foot and with a relative accuracy (linearity) of ±1%; (4) that the mechanical unit automatically move the transducer through a predetermined position sequence and transmit to a recorder the radial distance from the transducer to the bottom and to the position identification; (5) that the 6-inch piezoelectric transducer transmit a beam of ultrasonic energy not exceeding 3° in width (see fig. 6); (6) that the mechanical unit be designed for submersion to a depth of 25 feet; and (7) that a rail, permanently attached to a bridge pier, be provided to lower and raise the mechanical unit.

The requirement for a maximum range of 150 feet necessitated an ultrasonic frequency low enough to avoid excessive water losses, and the requirement of a maximum beam width of 3 degrees with a maximum transducer diameter of 6 inches limited the frequency of operation to a minimum of 200 kHz. This frequency was found suitable for the 150-foot range requirements and was used in all subsequent testing and design.

Mechanical unit

The mechanical unit contains the transducer, the drive mechanism, and the automatic sequencing circuits. Because this unit operates under water, its case was designed to resist the impact of water-borne debris and

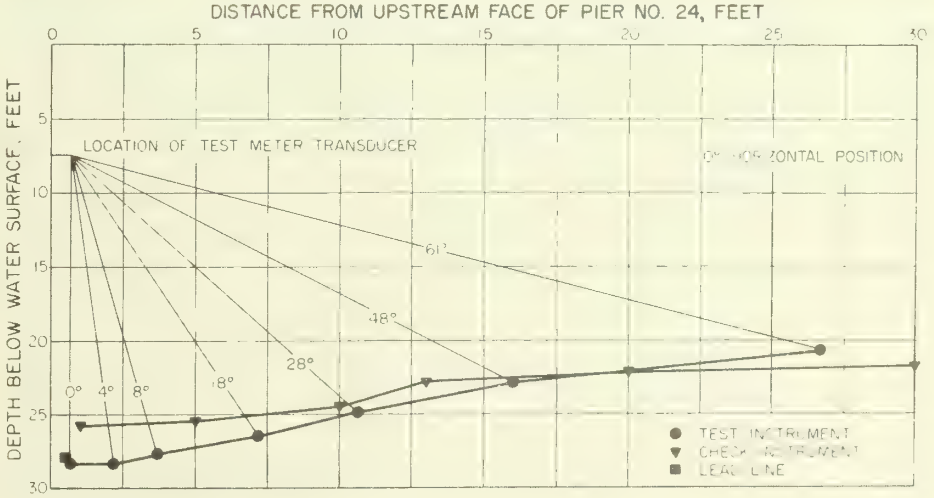

Figure 1.—Depths recorded during test run No. 3.

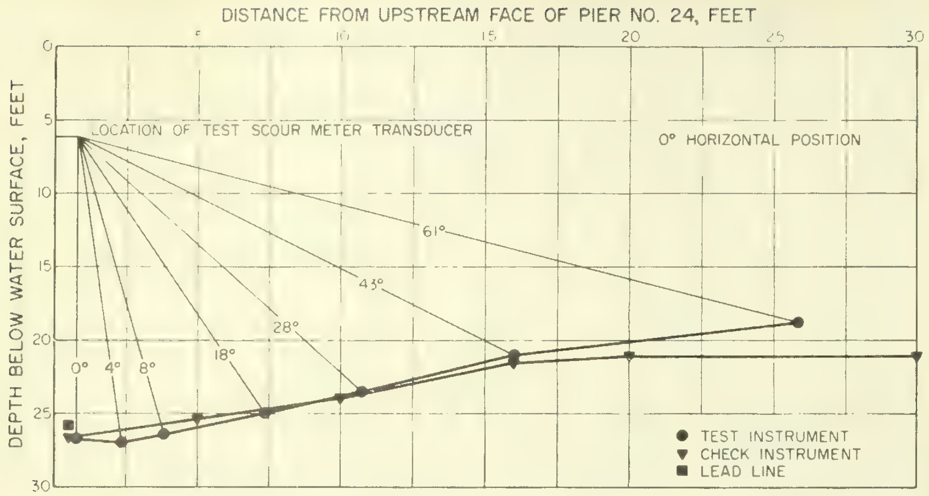

Figure 5.—Depths recorded during test run No. 5.

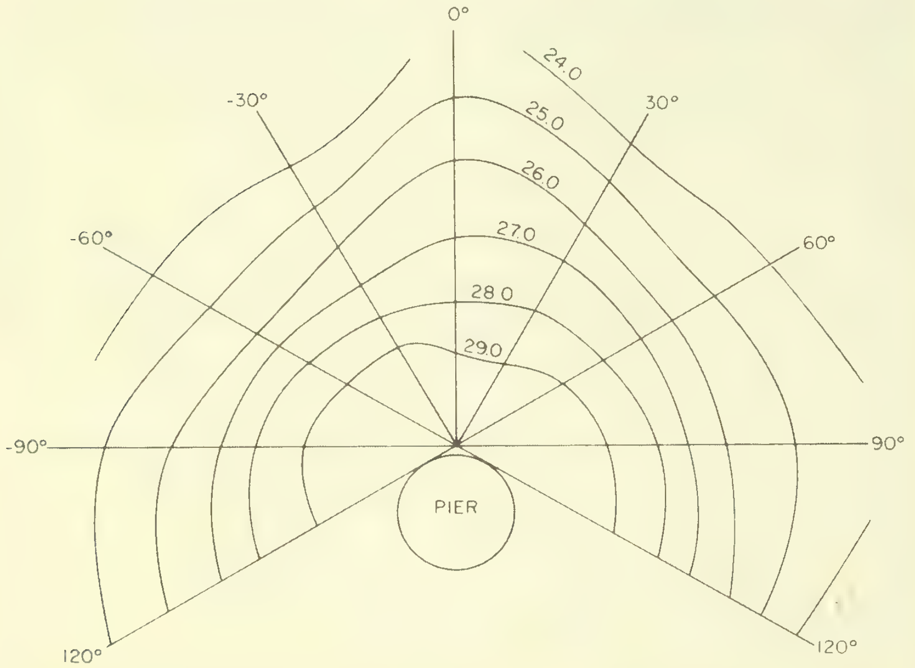

Figure 6.—Scour map depth contours (number of feet below water surface).

was constructed of 3/8-inch thick stainless steel with guide loops welded on the back for installing the unit on permanent pier rails. Figure 8 is a photograph of the unit mounted on the pier rails. Complete, the unit weighs 85 pounds in air and about 30 pounds in water.

The sequencing and position indicating circuits are all contained in the mechanical unit. Automatic sequencing is accomplished by 6 relays, 12 microswitches, 20 diodes, and an RC timing network. Position information is provided by 9 diodes, 10 resistors, and a 10-turn potentiometer. The drive mechanism consists of horizontal and vertical drive gear assemblies and two 24-v.d.c. drive motors. The horizontal drive operates in one direction only and covers the full 360° movement. Microswitches, mounted just outside the circumference of the horizontal gear are activated by a cam on the gear to provide horizontal position information as well as automatic sequencing signals. The vertical drive operates in both directions over a 61° gear segment. The vertical drive mechanism, vertical position indicating potentiometer, and vertical sequencing cams and microswitches are all mounted on the horizontal gear. The 6-inch diameter transducer is connected to the vertical drive gear by a shaft that protrudes from the bottom of the unit.

Vertical position is provided by a voltage at the arm of the potentiometer coupled to the vertical drive mechanism. Horizontal position is indicated only when the horizontal gear is stationary at one of the incremental horizontal positions. Each discrete horizontal position connects to a point on a voltage divider network to provide a different voltage. The separate horizontal and vertical voltages are transmitted to the control panel of the electronic unit through the interconnecting cable. The two position-indicating voltages are then added to provide a distinctive position indication. Recorded radial distance and position data are shown in figure 9.

Electronic unit

The electronic unit is housed in a watertight case and comprises a power module, transmitter module, receiver/computer module, display/test module, and a control panel for the mechanical unit. A complete block diagram of the instrument is shown in figure 9,

The power module contains two 24-volt batteries and recharging circuits. Starting with fully charged batteries, the electronic unit may be operated for 24 hours if the positioning mechanism is not operated. Operating the positioning mechanism of the mechanical unit once every 10 minutes reduces the operating time to 8 hours.

The transmitter module contains a voltage regulator printed circuit board and a transmitter panel control board. The voltage regulator board provides regulated voltage, plus and minus 15 volts, for the operational amplifiers in the computer circuit and connects to the power switch on the front panel. The transmitter circuit consists of a unijunction oscillator, a clock flip-flop, a gated 200-kHz oscillator, a voltage amplifier, a power amplifier, and an output transformer. The

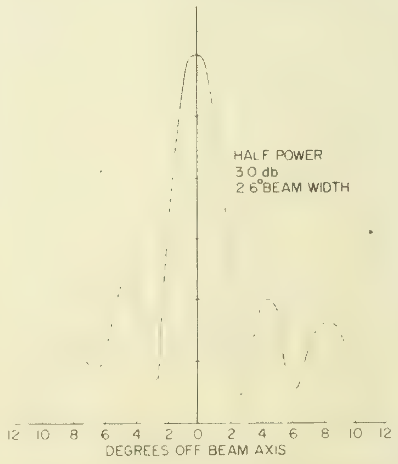

Figure 7.—Directivity pattern of 7-inch transducer.

output voltage to the transducer is approximately 100 volts peak-to-peak.

The clock repetition rate is controlled by the position of the range switch on the front panel of the receiver computer module. The rate is changed from a maximum of 30 pulses per second for the 100-foot position, to 6 pulses per second for the 150-foot position. The design requires that the round-trip time for a signal pulse be less than one-half the clock period. The round-trip time for this instrument is approximately 62 milliseconds, and the one-half clock period is approximately 80 milliseconds.

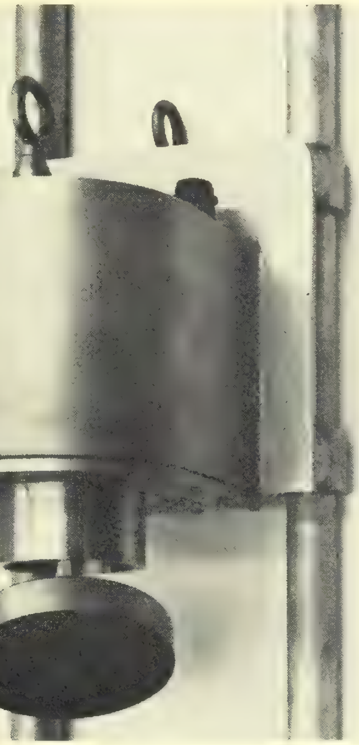

Figure 8.—Mechanical unit mounted on pier track.

The receiver/computer module contains the receiver printed circuit board, the computer printed circuit board, and the operating controls. The receiver circuit consists of an RF section, detector, and video amplifier. Included on the receiver panel control board is a gating circuit that makes it possible to eliminate all received signals originating at a distance less than the desired target distance. This circuit enables the operator to eliminate a large part of any undesirable output. The output of the receiver panel control board is a pulse which is equal in duration to the time required for the ultrasonic signal to travel from the transducer to the target and back again. The pulse is generated by a bestable multivibrator, which is turned *on* by the transmitted pulse and turned *off* by the first echo received through the receiver and gate circuits.

The computer consists of a ramp generator and a sample-hold circuit with appropriate switching and resetting circuits. The pulse from the receiver is clipped to a constant amplitude by a voltage reference diode and coupled to the ramp generator input. During the time that the pulse is present, the output voltage at the operational amplifier increases linearly. When the pulse ends, the voltage at the ramp generator output remains constant until the end of the first half of the clock square wave. The second half of the clock square wave resets the ramp generator and holds it in the reset condition until the start of the next clock cycle. Immediately after the input pulse ends, a sample pulse is generated. This sample pulse is approximately 2 milliseconds in length and connects the ramp generator output to the sample-hold input for the duration of the pulse. During the short time that the sample pulse is on the sample-hold, the output voltage is the same as the ramp generator voltage at the input. Between sampling pulses, the sample-hold circuit holds the last voltage value seen at its input during the sampling pulse time.

The output of the sample-hold operational amplifier is the output of the instrument. Thus, the output is basically a d.c. signal that changes only when the distance to the target changes. An important feature of the instrument is that if the return signal is momentarily lost (because of sediment, etc.), the output signal will drop toward zero rather than indicate full scale.

The display/test module contains a 2-inch oscilloscope; a d.c. to a.c. inverter to provide power for the oscilloscope; signal connections from the clock, transmitter, receiver, and computer boards; and a switch for the oscilloscope display. Although the module was intended to be used during normal operation, it was found that the high voltage inverter square wave caused too much interference when the sensitivity control was turned to a practical level. However, the test function of the module is useful for troubleshooting and checking the operation of the critical circuits. The transmitter pulse, clock square wave, time pulse, and ramp generator output signals

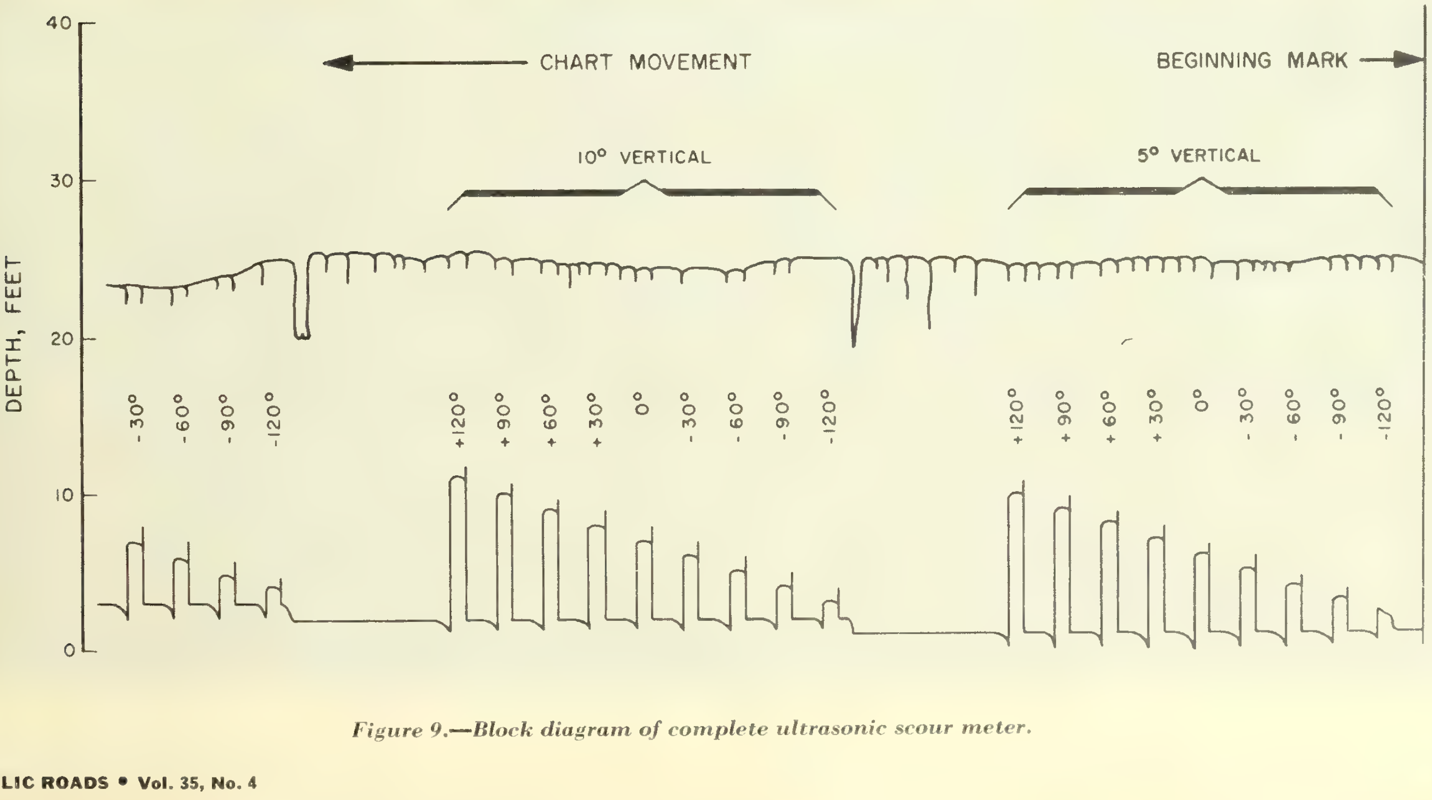

Figure 9.—Block diagram of complete ultrasonic scour meter.

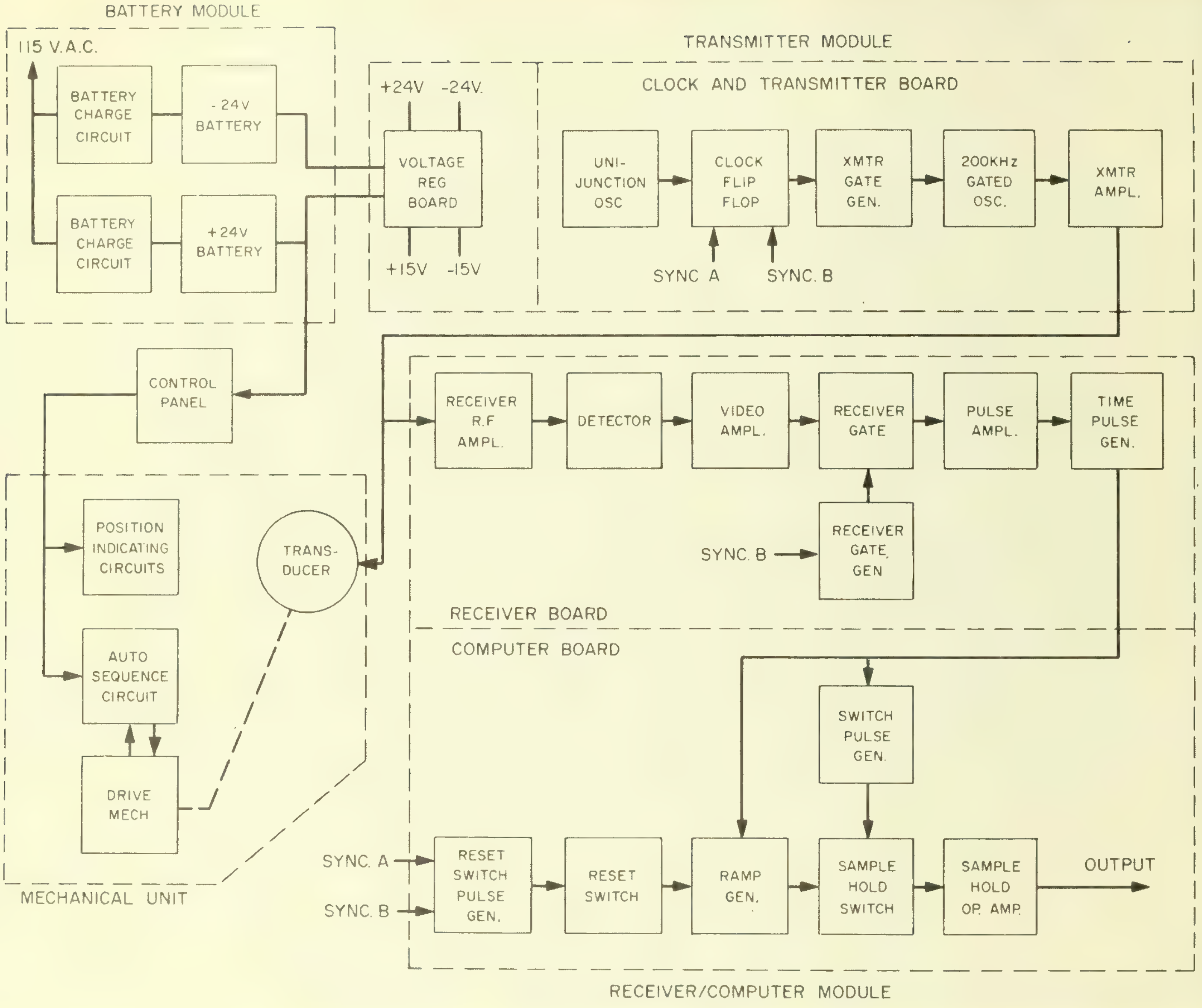

Figure 10.—Strip-chart recording of depth and position information.

are all switch selectable on the panel of the module.

The control panel connects to the mechanical unit to provide input voltage and output range and position signals. Output connectors provide a signal consisting of the two position signals added together as shown in figure 10. The push button for starting the sequence is located on the control panel. A push button and output jack, also on the control panel, provide remote marking capabilities.

Output signal and recording

The output signal, indicating depth, is a slowly varying d.c. voltage with a very low impedance. Although the basic instrument does not include a recording device, this output permits the use of a wide variety of recorders without adversely affecting the linearity of the instrument. The only restriction is that the recorder should require an input current of less than 1 milliampere. Currents in excess of this will not damage the equipment but will reduce its accuracy. Thus, any recorder used should have a minimum input resistance of 10 kilohms.

For most applications a two-channel recorder would be desirable one channel to record depth as a voltage level and another channel to record position information. On all ranges, as selected by the RANGE switch on the instrument, full scale accuracy and linearity is better than 1 percent. For applications in which data are to be recorded for future processing by computers, a digital voltmeter that provides a digital output can be used to advantage. The output signals from the digital voltmeter may be recorded for later processing using a tape recorder or a paper tape punch machine.

Instrument Utilization

The instrument described in this article was designed to obtain detailed scour data for research purposes. It will be used as a part of a large scale effort to collect data on the many factors that influence the local scour process.

The concept and basic instrument design could, however, be adapted for use in inspection or maintenance operations. In such applications, some reduction in accuracy and detail could be tolerated, and a less sophisticated design used. The mechanical unit described here must be complex to provide the large number of data points necessary to obtain a detailed scour-hole map. However, by reducing the number of data points to several in a

(Continued on p. 108)

Drivers' Decisions in Overtaking and Passing

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

The research reported in this article was concerned with how well drivers can judge the distance they require to overtake and pass another vehicle on the roadway. An experiment in which 20 drivers made estimates of overtaking and passing distances in their own car and in a Government car, at speeds of 18, 30, and 50 miles per hour, was carried out on an airplane runway. It was found that drivers are unable to estimate overtaking distances accurately, but rather tend to make dangerous underestimates, particularly when the car in front is moving rapidly. Applications of the research results are suggested for the design of highways and traffic control devices.

**Reported by DONALD A. GORDON and
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**Research Psychologists,
Traffic Systems Division**

Introduction

On a simple 2-lane rural highway, the driver of an automobile must often overtake and pass a car ahead to maintain a reasonable speed of travel. Even though the driver is the essential element upon whose judgment the safety of passing performance depends, little is known about his decision processes.

To overtake and pass another vehicle, the driver must carry out the passing maneuver in the time and space available. Accordingly, the maneuver can be analyzed in terms of four basic quantities:

- α is gap time or distance separating the overtaken vehicle and the vehicle in the opposing lane.
- α' is the driver's estimate of gap available.
- β is the time or distance required by the driver-car combination to perform the maneuver.
- β' is the driver's estimate of time or distance required to perform the maneuver. The two prime quantities involve psychological characteristics which are distinguished from physical variables.

The driver's judgment in overtaking and passing involves a comparison of α' and β'. If the outcome is favorable, that is, the gap available, α, is judged to be longer than the distance required, β, with adequate safety margin, the driver will accept the gap. If not, he will reject it and wait for a longer gap. With practice, the driver is able to make the gap decision rapidly.

The $\alpha\beta$ concept also applies to the driver's gap judgments in merging, in passing an intersection, and in other driving decisions as well. When the driver makes a U-turn, the width of the road, α, is related to the turning radius of the car, β. In parking, the driver compares the parking space, α, with the width of the car plus the room required to open the doors, β.

Both α and β are measured in physical units of time and distance. Subjective quantities, α' and β', are also measured in physical units, and they may be obtained through psychological experimentation. Silver and Bloom measured gap size, α', by asking drivers to indicate the distance when an opposing car was just 12 seconds away (*1*).[2] β' can be measured by having the driver indicate the minimum distance at which he can just perform the maneuver. Whether time or distance is used to measure the gap depends on the application. Time is the usual measure of intersection gaps (*2*, *3*), but both have been used in overtaking studies (*1*, *4*).

Previous Research

The literature on overtaking and passing has been reviewed by Farber and Silver (*5*). Early studies were concerned mainly with establishing performance norms for traffic control. Matson and Forbes (*6*) and Prisk (*7*) give figures on overtaking distance when the pass was started at the same speed as the car ahead (accelerative pass) and when the following car had an initial speed advantage (flying pass). A distinction is also made between voluntary (unhurried) returns to lane and those in which the overtaking car was forced to return by the oncoming car.

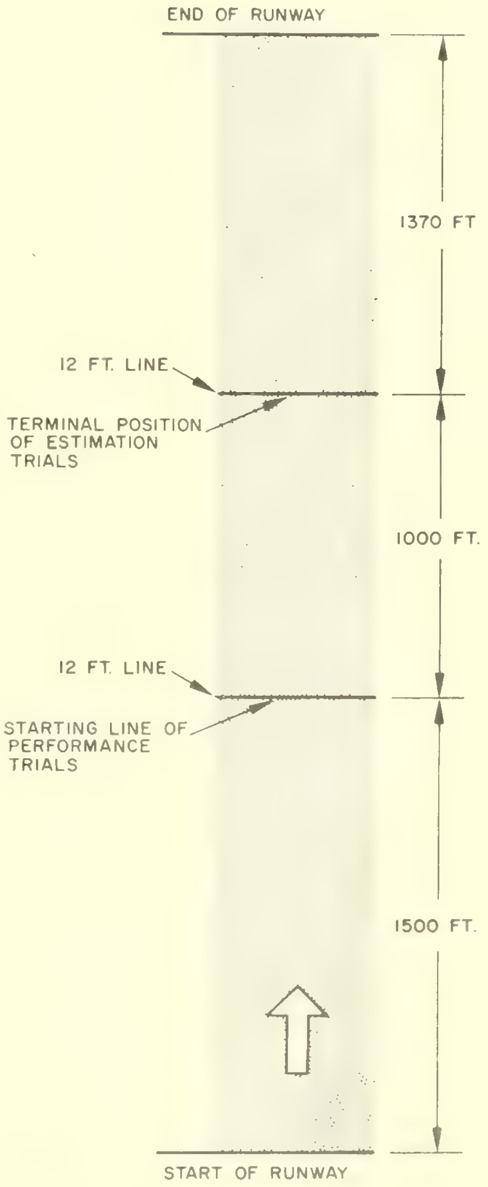
END OF RUNWAY
1370 FT
12 FT. LINE
TERMINAL POSITION OF ESTIMATION TRIALS
1000 FT.
12 FT. LINE
STARTING LINE OF PERFORMANCE TRIALS
1500 FT.
START OF RUNWAY

Figure 1.—The experimental track.

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[2] The italic numbers in parentheses identify the references listed on page 101.

The first psychological study of overtaking and passing was made by A. Crawford (*4*). Crawford thought of overtaking and passing judgments as psychophysical, and he carried out controlled experiments in which accepted gap distance, overtaking, and safety distances were measured. He also carried out a validating highway study in which overtaking vehicles were observed from the window of a light van. Crawford's findings on overtaking performance and safety distance will be discussed under *Results*.

Silver and Bloom showed that the driver could not make accurate α judgments. They instructed the driver to indicate when an oncoming car was just 12 seconds away, simulating a 12-second passing time, β. Without specific knowledge of the speed of the oncoming car, drivers gave their passing judgments at the same distance; but when they were told the speed of the oncoming car, they gave improved estimates of the 12-second-gap passing distance. Rockwell and Snider recently showed that the driver does make a limited use of oncoming car speed in making α estimates (*9*). The study reported here may be considered the converse of the Silver and Bloom study—α characteristics were simplified and standardized to test drivers' abilities to estimate β.

The need to consider the α and β characteristics of the overtaking decision was illustrated in a study conducted by Jones and Heimstra (*8*) in which drivers were told to indicate the last moment they could safely pass a lead car and avoid hitting an oncoming car. The drivers indicated the time, but did not actually pass. At the lead car speed of 60 m.p.h., 83 out of 190 judgments made during the study were shown to be unsafe; that is, the actual passing maneuver would have required more time than drivers gave it. The time required for the maneuver was determined in preliminary passing trials in which there was no opposing vehicle. Although Jones and Heimstra showed overtaking to be unsafe, their study does not tell whether drivers' errors were caused by the inability to assess the gap, α, by failing to estimate vehicular passing capability, β, or by both difficulties.

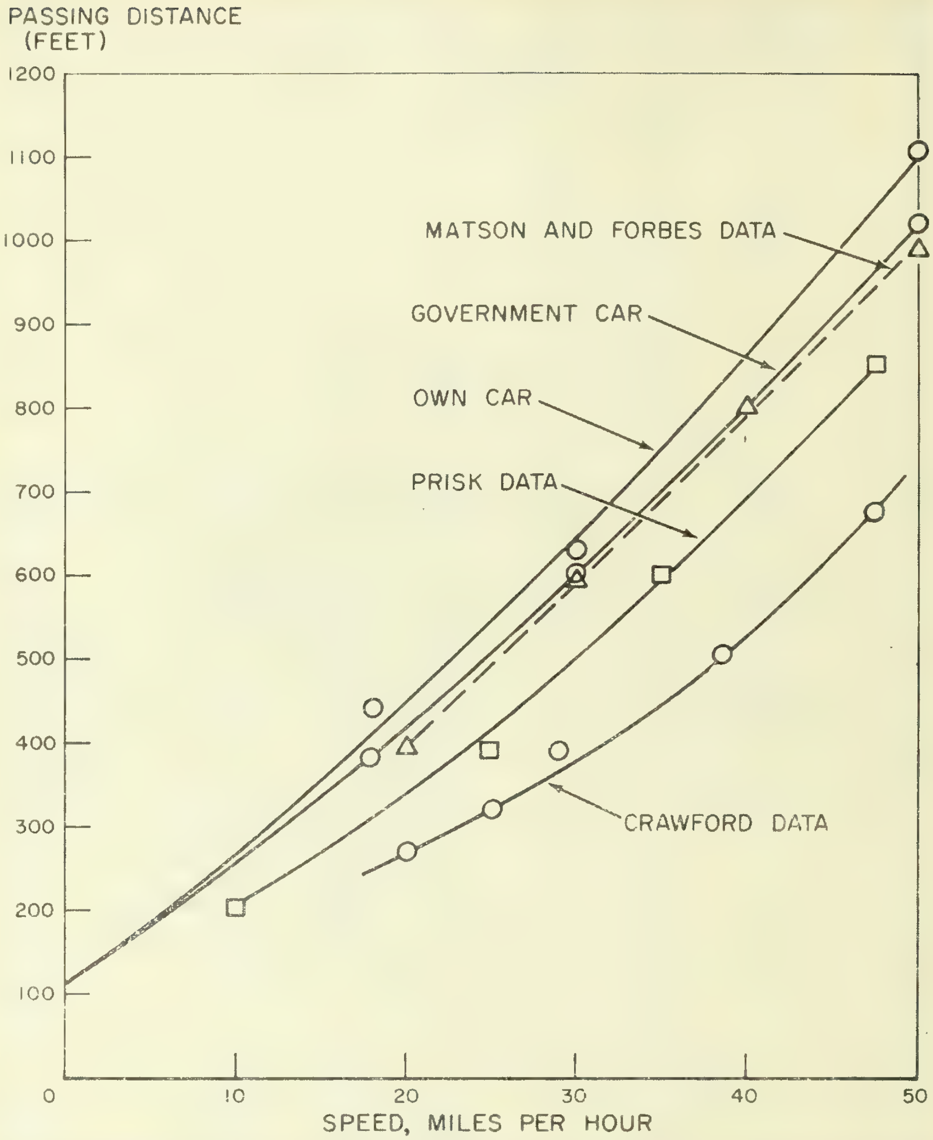

Figure 2.—Performance studies, overtaking and passing distances.

Research Reported Here

The research reported in this article was concerned with how well drivers can judge the distance they require to overtake and pass another vehicle on the roadway. Their estimations were simplified by terminating the maneuver at a fixed point along the road rather than by the passing of an oncoming car. In this way, errors in assessing the situation (α errors) were minimized. The drivers made the estimations in their own cars, and in another phase of the research, in a Government vehicle. These conditions indicate not only individual differences in performance, but also the effects of an unfamiliar vehicle.

Experimental track

The studies were carried out on the runway of the Beltsville Airport. (See fig. 1.) A 12-foot length of 2-inch reflectorized tape was placed across the driver's path, 1,500 feet from the start of the runway; another was placed 1,000 feet farther down the runway. The strips indicated to the driver the terminal position of the estimation trials and the starting position for performance trials. Each strip was made more conspicuous by a 12″ × 14″ white box positioned at the left margin. A numbered scale was laid out on the right edge of the runway.

Vehicles

In the first phase of the study, drivers operated a 1965 Government six-cylinder, 145-horsepower Plymouth sedan with which they were unfamiliar; in the second phase they used their own cars. No attempt was made to influence selection of the vehicles. All cars completed the tests except a 1959 Volkswagen, which could not overtake and pass another vehicle at 50 m.p.h. in the runway length.

The marking pistol

Positions on the runway where overtaking and passing occurred were indicated by a marking pistol (American Automobile Association detonator) attached to the rear bumper of each car. When the driver pressed a button, a solenoid release mechanism fired a shell containing yellow chalk at the runway. When the subject's vehicle did not have the 12-volt battery needed to activate the solenoid, the experimenter dropped a cloth marker to indicate runway position.

Drivers

The 20 drivers who served as experimental subjects were employees from neighboring university and U.S. Government offices. Drivers in the first phase included four males and seven females with ages from 20 to 52 years, and driving experience from 3 to 35 years; the median age was 23 years and the median driving experience was 7 years. Those in the second phase included eight males and two females with ages from 18½ to 46 years and driving experience from 2½ to 26 years; the median age was 20½ years and the median driving experience was 4¼ years. Drivers served 4 hours each and were paid $2 an hour for their work.

Method and Procedure

The overtaking and passing observations were part of a series of tests that also included braking and U-turns. The overtaking and passing procedure is described here.

Preliminary practice

Drivers drove to the end of the runway and back, twice, to familiarize themselves with the Government vehicle. Familiarization was eliminated in the second phase when drivers used their own cars.

Overtaking and passing estimations

Drivers followed the test car at a distance of 55 feet. They were instructed as follows: "You will follow the car ahead and think of passing it. When you come to the closest point to the line where you can still pass, using maximum acceleration of the car, indicate the spot by pushing the button." The distance between the lead car and subject car was maintained by the experimenter's instructions to slow down or speed up. The experimenter alined a taped spot on the windshield with the hood and rear bumper of the lead car to maintain the 55-foot distance. The speeds of 18, 30, and 50 m.p.h. were controlled by the driver of the lead vehicle. An experimenter stationed on the runway recorded the data. After each observation, the marking pistol was reloaded, and the lead and experimental cars driven to the starting point for the beginning of the next run.

Overtaking and passing performance

Performance trials were made after completion of the estimations. The driver followed the lead car at the scheduled pace. Instructions were as follows: "Follow the car ahead at the distance I tell you. When you get to the line, overtake and pass the car ahead, as fast as you can, and come back into the lane. Be sure you swing back into the lane." When the car was fully back in lane the experimenter in the test car pushed the pistol button. The experimenter on the runway then recorded the position of the chalk mark.

Three series of tests were run for both the estimation trials and the performance trials, the first series in each being a practice series. In each series, observations were made at speeds of 18 m.p.h., 30 m.p.h., and 50 m.p.h. Estimation trials followed each other without interruption, as did the later performance trials. The entire work was completed in a half-day, after which the driver was paid and dismissed.

Results

Performance β

Performance results are given in table 1. Standard deviations are maximum likelihood estimates. The variable error in the table is the mean deviation from the average of the two performances by each driver at each speed. Results are presented in graphical form in figure 2. Each point in the figure for Government car and own car represents the average of 20 observations. The zero point of 106 feet is the minimum distance required to pass a vehicle parked 55 feet in front of the starting line. The Matson and Forbes, Prisk, and Crawford data were presented for comparison.

The performance curves indicate that as speed increases, passing distance also increases but at an increasing rate. The least squares fit to the own car data is:

$$D=112.2+15.2\,V+.093\,V^2$$

Where

D = overtaking distance in feet
V = velocity in miles per hour

Performance of drivers using the Government car does not differ significantly from that of drivers using their own vehicles. The Matson and Forbes data points fall close to the Government car curve; the Prisk data have the same general form, but distances are about 100 feet less. Matson and Forbes and Prisk defined passing distance as car travel in the left lane, which is shorter than the passing distance defined here—from initial driver reaction to return to lane. Crawford's curves show still shorter distances, which perhaps is explained by his use of trained drivers and by other procedural differences. A complete analysis of these performance curves would consider vehicular accelerations at various speeds, the driver's willingness to use accelerative capacity of the car, and the driver's varying requirements for safety distance.

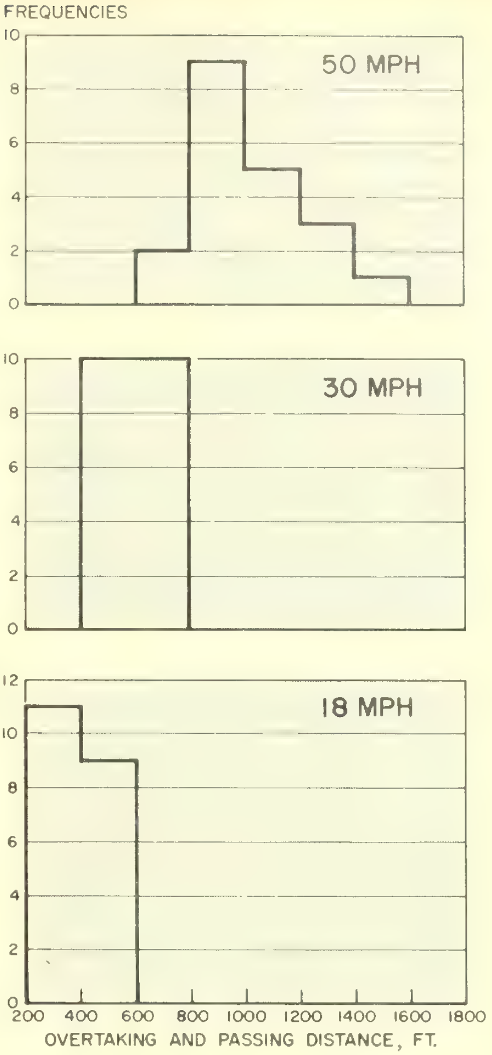

Figure 3.—Performance, overtaking and passing distances—Government car.

Drivers differ in their ability to pass, as indicated by the distributions plotted in figures 3 and 4. (See also standard deviations in table 1.) These differences were evident even when drivers used the same Government car. At 18 miles per hour, driver AR overtook in 284 feet, but driver GR required 455 feet. At 30 and 50 miles per hour, variability was larger than at 18 miles per hour. Some causes of these individual differences have been pointed out above. Drivers differ in reaction time, in willingness to use maximum acceleration of the vehicle, in safety distance requirements, and in the paths they take in returning to lane at the end of the maneuver.

The vehicle driven had more effect on passing distance than the driver who performed the maneuver. Variance in the own-car condition was significantly larger than in the Government-car condition in which the same automobile was used (f-test, .05 level, all speeds). Residual variance of own car

Table 1.—Performance, overtaking and passing distances

Performance	18 m.p.h.		30 m.p.h.		50 m.p.h.	
	Government car	Own car	Government car	Own car	Government car	Own car
Overtaking and passing performance:						
Mean feet	385.9	440.3	606.1	628.5	1,023.5	1,110.8
Standard deviation feet	53.7	76.7	77.9	121.3	192.8	289.2
Standard deviation/mean feet	0.139	0.174	0.129	.193	0.188	0.260
Variable error:						
Mean feet	12.3	22.0	27.0	32.7	47.3	41.7
Standard deviation feet	9.8	20.2	15.4	19.9	50.8	34.6
Variable error as percentage of performance percent	3.19	5.00	4.45	5.19	4.62	3.76

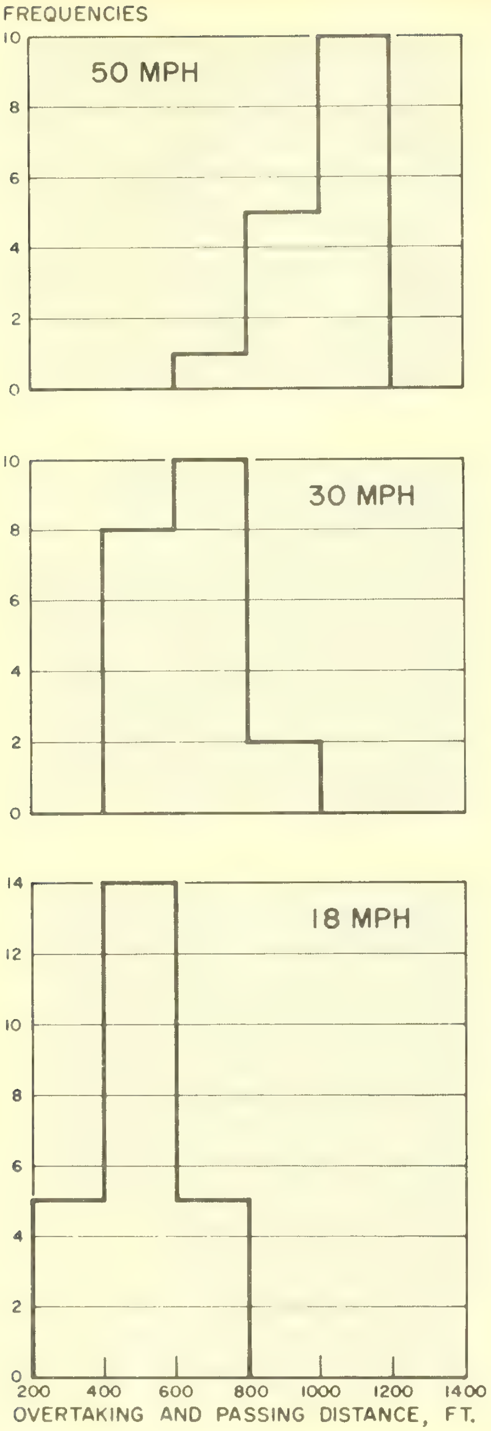

Figure 4.—Performance, overtaking and passing distances—drivers' cars.

Table 2.—Errors, overtaking and passing estimation

Errors	18 m.p.h.			30 m.p.h.			50 m.p.h.		
	Government car	Own car	Other	Government car	Own car	Other	Government car	Own car	Other
Constant error [1]									
Mean feet	197.2	136.9	...	312.2	129.6		317.9	237.5	.
Standard deviation feet	179.5	111.9	.	205.5	120.5		208.7	176.1	.
Variable error									
Mean feet	42.5	40.8		52.7	63.6		29.9	50.9	..
Standard deviation feet	30.5	33.4	.	40.1	41.8		14.4	44.7	.
Constant error/variable error	4.63	3.36		5.92	2.04	.	10.63	4.67	
Constant error/overtaking performance	0.511	0.309		0.515	0.206	...	0.311	0.214	....
Underestimation errors:									
Number	4	2		7	7	...	12	14	...
Percent	20	10	.	35	35	.	60	78	....
Crawford Interpolated Data percent	...	...	7	..	...	39	...	...	76

[1] Sign of the error, plus or minus, has been disregarded.

minus Government car is larger than Government car variance at all speeds. These variance calculations involve squared standard deviations of overtaking and passing performance in table 1. The importance of vehicular effects can also be seen in the individual records. The difference in performance between the highest powered car, driven by a 46-year-old woman, and the minimum powered car, driven by a 19-year-old boy, was larger than any set of driver differences on the same (Government) vehicle. These vehicular and individual differences relate to the groups studied and do not necessarily apply to the universe of cars and drivers on the road.

Drivers' errors

The errors made by drivers are analyzed in table 2. Constant error, listed for each speed, is the difference between each estimate and the mean of the two performances by the driver at that speed, averaged over all the drivers. Variable error is the deviation of each driver's constant error from his mean constant error, averaged over all the drivers. The underestimation errors occur when the constant error is negative. Frequency distributions of errors made in the Government and own car are plotted in figures 5 and 6. Each chart includes the 20 errors made at a particular speed.

According to the constant-error data in table 2 and the charts in figures 5 and 6, drivers are not able to estimate passing distance accurately. Constant error varies from about a fifth to a half of actual overtaking distance (table 2), and exceeds variable error at all speeds at the 0.01 significance level. It seems that drivers estimate their overtaking performance consistently but erroneously.

Drivers predict their overtaking performance better in their own cars than in an unfamiliar (Government) car, according to the constant error data, table 2. Errors in own car are significantly smaller than in the unfamiliar car (.05 sig. level, Fisher combination of experiments statistic, (*10*)). This finding implies that estimating vehicular performance, β', may be a learned aspect of driving skill. The finding also suggests that the driver's ability to estimate requirements for braking, U-turns, parking, and car following, may be a useful measure of his skill and effectiveness. Little is known about the nature of driving skills.

Negative estimating errors involving underestimation of maneuver distance are dangerous. Negative errors occur at all speeds, but are particularly frequent at high speeds. (See table 2 and figs. 5 and 6.) At 50 miles per hour, 60 percent of Government-car estimates and 78 percent own-car estimates would have been dangerous in the operational situation. The finding that underestimation is most frequent at high speeds, where accidents are most serious, is in close agreement with Crawford's results. (See table 2.)

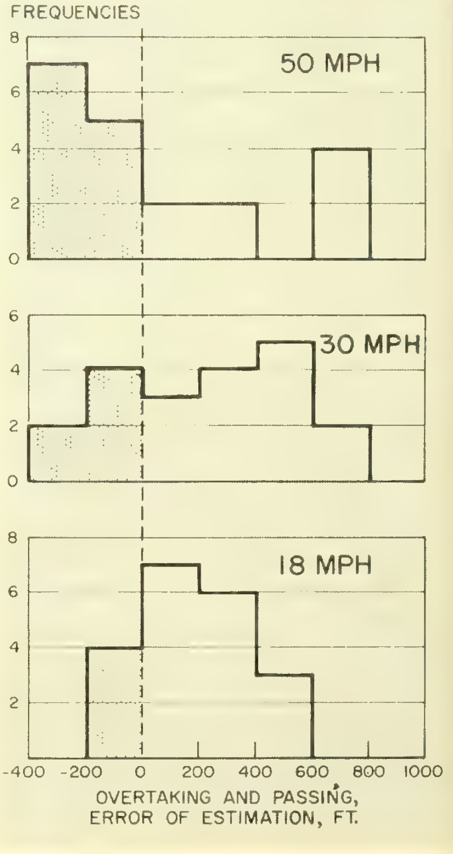

Figure 5.—Estimation errors in overtaking and passing—Government cars.

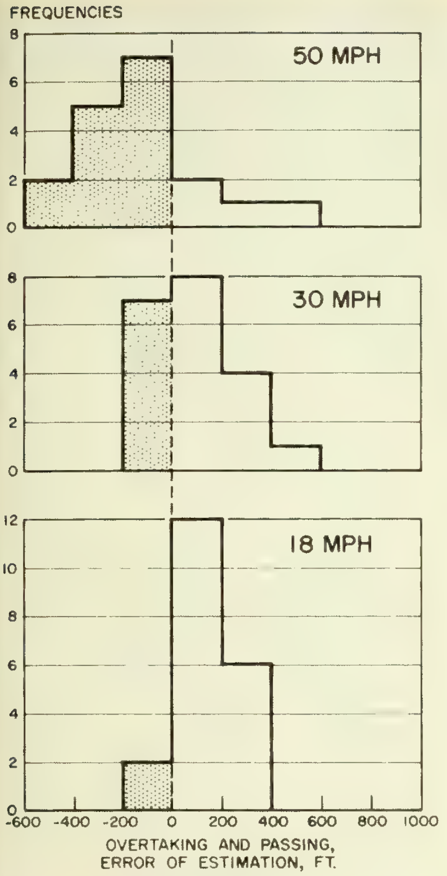

Figure 6.—Estimation errors in overtaking and passing—drivers' cars.

Driver's errors and underestimations in overtaking and passing may perhaps be explained by the difficulty of the judgment. Overtaking distance varies in proportion to speed, and there are as many overtaking distances as vehicular speeds. The driver cannot perform by simply learning a fixed distance, as might be the case in U-turns, or parking. The underlying speeds, accelerations, and distances are themselves subject to estimation error. For example, at 50 miles per hour, overtaking requires about 1,000 feet. An opposing vehicle, coming towards the driver at the same speed, would be twice as far away when the decision was made. The driver cannot be expected to make precise spatial judgments at such large distances.

The precise cause of underestimations at high speeds is not known. One explanation might be that the driver is not fully aware of how performance requirements, β, increase with speed, and he may continue to act as he did at slower speeds. Whatever its cause, high-speed underestimation remains a pertinent fact that highway engineers must contend with in dealing with the overtaking and passing maneuver.

Nonetheless, overtaking and passing accidents are not very frequent, and this fact perhaps requires explanation. Several safety factors are inherent in the situation to aid the driver. The driver may avoid danger by not passing at high speeds, and he may insist on an adequate safety distance. If a wrong decision is made, he may drop back into his lane, and the overtaken and oncoming cars may slow down and move to the shoulder. Traffic controls such as passing zones and signs also aid the driver.

Summary

In the study reported, driver's estimations of overtaking and passing distance were compared with actual overtaking and passing distance. An experiment, in which drivers made estimates in their own (familiar) car and in a Government (unfamiliar) car at speeds of 18, 30, and 50 miles per hour, was carried out on an airplane runway. On the basis of the results, the following conclusions are reached:

- Drivers are unable to estimate overtaking and passing distances accurately. Mean error ranges from 20 to 52 percent of performance distance. Significantly larger errors are made in an unfamiliar car than in the driver's own vehicle.
- Negative errors of underestimation, where the maneuver required more space than judged, increase with speed. At 50 miles per hour, 60 percent of the estimates made by drivers in the Government car and 78 percent of those made in drivers' own cars are underestimations.
- Overtaking and passing requires proportionally more distance as lead-car speed increases. The curve relating overtaking and passing distance to lead-car speed is well fitted by a second degree equation. These results are in agreement with those of previous investigators.
- Vehicular differences affected passing distance more than did driver variance in the groups studied.
- It is suggested that the driver's decisions in overtaking and passing can be studied in terms of four basic quantities: α, the gap time or distance available for overtaking and passing; α', the driver's estimate of gap available; β, the time and distance required by the driver to perform the maneuver, and β', the driver's estimate of time or distance required.

Applications

The finding that the driver is unable to estimate accurately his overtaking and passing requirements and that underestimations are frequent at high speeds implies that the overtaking and passing maneuver requires guidance. Possible aids that can be given to the driver are as follows:

- Passing areas, and *no passing* signs—traditional aids to overtaking and passing.
- Speed limits and other speed regulation, particularly in passing zones.
- Driver education emphasizing no passing at high speeds and cooperation with overtaking drivers.
- Road design modifications such as wibe shoulders and addition of lanes.
- Traffic planning to minimize use of 2-lane rural roads.
- Electronic devices to inform the driver when it is safe to pass. Such devices are currently under development in the Traffic Systems Division, Office of Research and Development, Bureau of Public Roads (*11*).

The $\alpha\beta$ concept provides a theoretical framework that may be useful in studying driver decisions in intersection and merging gap acceptance, and in such maneuvers as U-turns, braking, parking, and car following. The β' β comparison may be useful as a measure of driving effectiveness in the study of learning and driving performance.

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Effective and efficient highway maintenance is the objective of highway management research—one of the important facets of the Bureau of Public Roads' national program for research and development in highway transportation.

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Highway Maintenance Management Research—An Overview

**Reported by[1] WILLIAM N. RECORDS,
Highway Research Engineer,
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Introduction

DURING the past 50 years, highway maintenance management in the United States has experienced many changes. Most of them transpired through a process of gradual evolution and were based on intuition and pragmatic considerations, rather than on factual knowledge and scientific management principles. This process proved to be reasonably satisfactory between 1920 and 1949. Most maintenance organizations were satisfied with the status of their management and had no real reason to adopt a more sophisticated procedure. Therefore, it is not surprising to discover that maintenance management research was of little consequence during these years. Studies were few in number, limited in scope, and uncoordinated. Most of them were carried out informally, making it difficult to even document their existence. It would definitely be misleading to say that these efforts had a significant impact or that they constituted any kind of research program.

Beginning of Organized Highway Maintenance Management Research

In June 1950, an event signaled the beginning of an organized, formal maintenance management research program in the United States. The event was the initiation of the Connecticut Maintenance Study—a joint venture of the Bureau of Public Roads and the Connecticut State Highway Department. The objectives of the study were to develop basic facts concerning the performance of labor and equipment on field operations and to appraise management problems. Study results indicated that there were deficiencies and problems associated with field operations and there was a need for continued research. Prophetically, the report stated: "The further development and extension of the groundwork encompassed by this study can lead to the establishment of units of work and standards of maintenance accomplishments, thus making possible the estimation of labor and equipment requirements to perform the maintenance obligation in a particular area under certain given conditions."

The maintenance research program continued during the 8 years that followed the Connecticut Study, primarily because the Bureau of Public Roads retained an interest in it. About 20 small-scale studies were conducted on the field operations of State maintenance organizations. Results were not extensive enough to fully delineate management problems, but they did serve to verify two hypotheses: (1) Results of the Connecticut Study provided a good view of the situation in other States, and (2) many management problems were similar, varying only in degree from organization to organization.

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[1] Presented at the Maintenance Management Workshop, Highway Research Board, Ohio State University, Columbus Ohio.

In 1959, the program received an impetus when the Bureau of Public Roads joined the Iowa State Highway Commission to conduct a study that was considerably larger in scope than the one in Connecticut. The Iowa maintenance study was primarily designed to produce facts that could be used by management to control and improve the economy of maintenance operations. It included collection of basic data concerning the performance of labor and equipment on field operations, variations in total workloads and work units, utilization of supervisory personnel, and other aspects of maintenance management. Considerable emphasis was placed on data analysis to pinpoint problems and develop possible solutions. In a few operations, proposed solutions were tested for practicality.

The Iowa Study received considerable publicity during 1960 and 1961. Its findings were accepted by many maintenance managers as indicative of the situation in their own organizations. At approximately the same time, other forces began operating to change managers' attitudes toward management and management research.

Maintenance Problems Promote Research

Between 1960 and 1967, maintenance organizations in this country were subjected to a number of external pressures that caused severe internal stresses. Among the most intense pressures were those due to:

- Addition of new facilities on the Interstate and other systems.
- Public demands for higher levels of maintenance.
- Rapid changes in the technology of highway design, materials, and equipment.
- A labor market which could not supply an adequate number of qualified personnel.
- Constrictions on maintenance budgets to make the maximum amount of funds available for badly needed construction projects.
- Campaigns to tighten up the fiscal and administrative control of highway organizations.

The type of management that had been tolerated for many years simply was not able to cope with the stresses induced by these pressures. Problems multiplied and managers became painfully aware of what was happening. Many concluded that their organization's management was deficient and needed improvement. As attitudes toward management changed, there was increased interest in all kinds of management research. The organized, formal program expanded considerably between 1960 and 1967, with studies covering a wide variety of subjects.

Three studies which related to costs for maintaining specific types of roads were conducted in Louisiana. These studies were aimed at developing a procedure for estimating costs through the use of mathematical regression models based on historical fiscal records. An Idaho study had a similar objective and used a like procedure. The Oklahoma Department of Highways also carried out a study of maintenance costs for specific types of roads. They used a technique that considered both historical fiscal data and deterioration ratings for selected test sections.

A major study of maintenance costs, conducted for the National Cooperative Highway Research Program, was primarily directed toward developing a method of predicting Interstate highway maintenance requirements. The technique used was similar to those for the aforementioned State studies. Cost data from selected test sections across the nation were analyzed to develop mathematical models for seven groups of maintenance activities that could be used to predict *requirement units.* A secondary objective of the study was to develop a new maintenance expenditure index.

Three research studies were conducted on roadside activites. In Louisiana, the State University identified optimum equipment and work methods for mowing highway roadsides. In Indiana, the Highway Commission emphasized development of comparative costs for different types of roadsides and mowing methods. In Ohio, the State University identified the most effective means of caring for Interstate roadsides.

In Illinois, New Jersey, New York, and Wisconsin, a series of studies was conducted for State Highway Departments and Toll Road Authorities. The studies were designed to establish long range requirements for major maintenance on high type facilities. In conjunction with detailed field inspection of facilities, they utilized new techniques for predicting pavement deterioration.

The objective of a major study in Los Angeles was to develop a program that would improve the planning, directing, and controlling of labor and equipment assigned to various field activities. An important feature of this study was the use of the industrial engineering technique, Methods-Time Measurement, to analyze each activity and to develop standards for work methods and performance. A similar project took place in San Diego County.

A Minnesota study utilized industrial engineering techniques to develop improved work methods, establish performance standards, and improve the maintenance reporting system. A smaller, but similar study was conducted in New Jersey.

The largest single study was carried out by the Virginia Department of Highways. This effort lasted 3 years and was designed to cover nearly every major aspect of maintenance management. It involved the collection of data concerning performance of labor and equipment, development and testing of improved work methods; establishment of quality, quantity, and performance standards; development and testing of a new reporting system; development and testing of a budgeting system; development and testing of training material; and work in several other areas.

The cited studies were started, and essentially completed, during the 8-year period ending in June 1968. They indicate the extent and scope of activities during this period but do not account for the entire research program. Nine other studies, started between 1960 and 1967, were not included because they are still active and will be described later. A few other studies have been omitted here because of their limited scope.

One other aspect of the completed studies deserves mention. More than half were funded through the Federal-aid Highway Planning Research Program (HP&R). This program provides for joint State-Federal financing of research in areas that have a significant influence on highway transportation in the United States. For many years, maintenance management has been recognized as one of these research areas. In 1964, this position was emphasized when a project for maintenance operations and management was included in the 27 top priority projects of the Bureau of Public Roads National Program for Research and Development in Highway Transportation. Partly as a result of this emphasis, annual Federal-aid expenditures in the maintenance area have more than doubled in the last 5 years.

Current Maintenance Research Program

Currently, the maintenance management research program includes 13 fully active studies. Eleven of these are being financed through the Federal-aid HPR program. Their estimated total cost is more than $2 million, and annual expenditures are about $700,000. The other two studies are financed entirely with State funds. Their estimated total cost is more than $500,000, and annual expenditures are about $150,000. The nature and scope of these studies vary considerably; six can be classified as comprehensive because they cover several aspects of maintenance management, five deal with the equipment and methods for specific activities, and two are concerned with costs. Summaries of pertinent facts about each study are presented in table 1. Other current informal research efforts, such as those underway in Illinois, New Jersey, and New York, may ultimately become a part of the formal program.

Some of the significant results that have emanated from these efforts are as follows:

- The research has clearly demonstrated that in most highway maintenance organizations management is beset by a number of problems including inadequate factual data concerning field activities; nonuniform standards or a lack of standards; ineffective procedures for planning and scheduling work; widely varying quality, productivity, and unit costs for field activities; ineffectual means of comparing actual and desired quality, service, level, and unit cost for maintenance activities; lack of a reliable means to forecast long range maintenance requirements; lack of a means to evaluate alternative policies; and shortage of trained personnel.
- New systems of maintenance field reporting have been developed and proven capable of supplying the kind of information

Table 1.—Summary of current maintenance management research studies

Study title	Agency	Estimated Cost	Period	Objectives
		Dollars		
Maintenance practices	Ark. State Highway Department (HP&R) [1]	169,000	July 1967–July 1972	Evaluate the maintenance accounting system and revise as needed Define maintenance standards Evaluate existing maintenance practices and develop improved practices Identify training needs; develop and test training materials
Maintenance management	La. Department of Highways (HP&R) [1]	575,000	Sept. 1965–July 1969	Define responsibilities and functions for various management levels Evaluate training needs; develop and test training materials Develop and pilot test a maintenance work reporting system Determine the most effective methods and staffing for maintenance activities Establish maintenance standards for quality, quantity, and productivity Develop and test an overall maintenance management system
Comprehensive maintenance	N.C. State Highway Commission (HP&R) [1]	220,000	July 1966–Dec. 1972	Determine the adequacy of the maintenance organization to carry out its assigned responsibilities and functions Evaluate the present maintenance management system with emphasis on reporting Determine the relationship between maintenance costs and factors such as traffic Determine major maintenance operations whose efficiency and economy can be improved; develop improved methods, etc. Evaluate maintenance facilities and materials
Highway maintenance management	S. Dak. Department of Highways	300,000	July 1968–Oct. 1970	Develop and test quality, quantity, and productivity standards for maintenance activities Develop and test a maintenance work reporting system Develop and test a maintenance work scheduling process Develop and test a maintenance budgeting process Develop a methods and training unit Conduct a performance laboratory to test developments
Maintenance management	Utah State Road Commission (HP&R) [1]	285,000	Apr. 1967–Aug. 1969	Establish quality, quantity, and productivity standards for maintenance activities Develop and test a maintenance work reporting system Design, develop, and test an overall maintenance management system Evaluate the field organization and resource utilization Prepare a plan for improving maintenance performance
Maintenance improvement program	Wash. State Highway Commission	250,000	July 1967–Dec. 1968	Develop standards for measuring performance of maintenance operations Establish procedures for maintenance planning and scheduling Provide data for improved maintenance budgeting and control Train maintenance supervisors
Tunnel cleaning method	Calif. Division of Highways (HP&R) [1]	117,000	July 1967–June 1972	Develop a tunnel cleaning method that is rapid, economical, nonhazardous, and nondestructive
Cost effectiveness of antiskid and deicing programs in Pennsylvania.	Pa. Department of Highways (HP&R) [1]	20,000	July 1968–June 1970	Study and evaluate existing snow and ice control practices Develop improved methods, equipment, and materials for snow and ice control
Winter maintenance for bituminous pavements.	Tex. Highway Department (HP&R) [1]	36,000	Sept. 1967–Aug. 1969	Evaluate existing practices for winter pavement maintenance Develop improved methods, equipment, and materials for winter pavement maintenance
Snow removal and ice control techniques at interchanges.	NCHRP [2]	50,000	Sept. 1967–July 1969	Identify and evaluate the factors which influence the efficiency of snow removal and ice control operations at interchanges Develop operational systems that will provide for efficient snow removal and ice control procedures on interchanges in both rural and urban locations
Develop performance budgeting system to serve highway maintenance management.	NCHRP [2]	250,000	Sept. 1968–Oct. 1970	Develop and test a model highway maintenance budgeting system
Maintenance formula application	La. Department of Highways (HP&R) [1]	60,000	July 1963–July 1969	Accumulate accurate cost data for testing and revising mathematical models to predict maintenance costs
Maintenance cost	Ohio Department of Highways (HP&R) [1]	380,000	July 1961–July 1972	Determination of reliable maintenance costs Determine the influence of major factors which contribute to maintenance costs Measurement of the level of maintenance and determination of the extent to which deficiencies exist in the current maintenance and operation of the highway system

[1] HP&R—Highway planning and research project (Federal-aid).

[2] NCHRP—National cooperative highway research program.

needed for management, fiscal accounting, and research purposes.

- Quality, quantity, and performance standards for maintenance activities have been developed and proven practical for operational use.
- Techniques for planning and scheduling maintenance activities on long range and daily bases have been developed and proven practical.
- A large fund of data concerning work methods, time utilization, and productivity of labor and equipment under typical field conditions has been accumulated and analyzed to determine some cause-effect relationships.
- Procedures for determining optimum staffing patterns and work methods have been developed.
- New equipment and methods have been developed and proven practical and economical for several maintenance activities.
- Performance budgeting systems for maintenance are being developed and tested for practicality.
- Procedures and data for forecasting long range maintenance requirements have been developed and proven useful.
- Techniques to enable management to evaluate alternative policies for investments, resource allocation, staffing, and other aspects of maintenance are being developed.
- Some materials and procedures for training maintenance personnel have been developed and tested.

At the present time it would be unrealistic to state that these research results have been widely translated into improved management. The current attitudes of administrators and managers have created a favorable climate for their use, but unfortunately, each maintenance organization must operate under its own set of conditions and constraints. Thus, it is usually necessary to undertake some additional research to adapt results for a particular situation. This takes time. Still more time is required for actual implementation. Hopefully, a significant number of organizations will soon begin this process, but it will be at least 5 years before there is a widespread improvement of maintenance management in the United States.

Finally, everyone involved in maintenance management research needs to recognize that the objective is not simply improved management. This is only the key that unlocks the door leading to the ultimate goal—effective, efficient maintenance for our nation's highways.

Criteria for Highway Tunnel Lighting Design

(*Continued from p. 90*)

ing installation and a tunnel is usually the dirtiest part of any highway.

To use a *maintenance factor* in designing lighting for tunnels is to admit that no amount of physical maintenance can keep the output of the lighting system up to its initial level. The value of the maintenance factor used indicates the amount of the uncontrollable depreciation that is expected and the amount of effort that should be devoted to overcome this depreciation. After the maintenance program is established, the designer can determine an appropriate maintenance factor for the system. A maintenance factor range from 0.25 to 0.60 would be typical, depending on the proposed maintenance schedule.

Approach roadways

Current lighting criteria should govern the design of any roadway lighting, and the presence of a tunnel will not necessitate any change in the criteria. If lighting is not installed on the approach roadways, a few hundred feet of transition lighting leading to and from the tunnel may be used. When no roadway lighting exists the night lighting in the tunnel should be designed to a minimum acceptable level to avoid abrupt lighting transitions.

For daytime conditions, the roadway approaching the tunnel can be designed to aid the road user's visibility by reducing the adaption level. As the approach to the tunnel will be either a detriment or an aid to the road user's visibility, it is recommended that this area have a low reflectance finish, including dark portal face and dark pavement surface.

ACKNOWLEDGMENTS

The authors wish to acknowledge the contributions of the following persons whose unpublished papers and reports provided background material for the preparation of this article:

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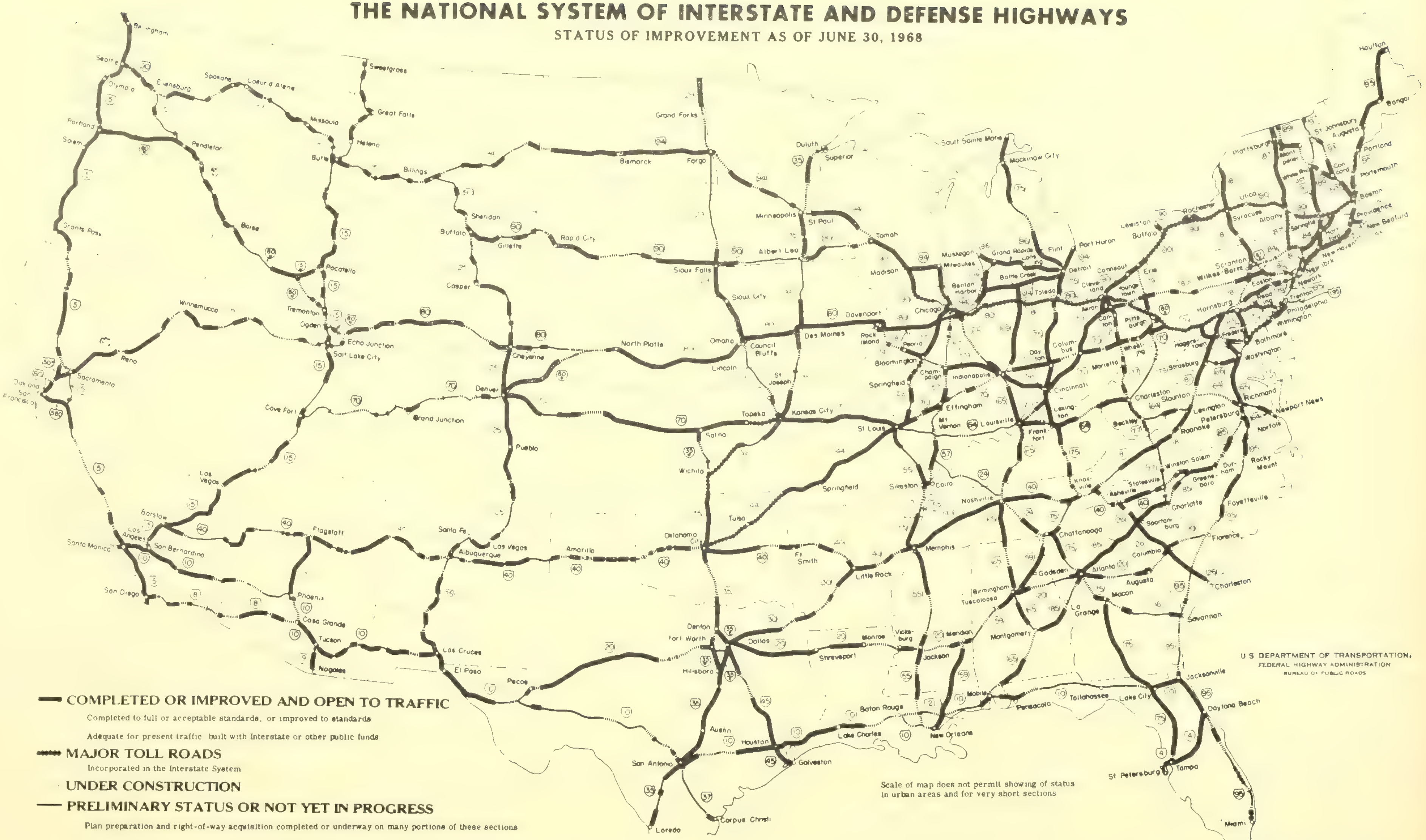
THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF JUNE 30, 1968



COMPLETED OR IMPROVED AND OPEN TO TRAFFIC

Completed to full or acceptable standards, or improved to standards

Adequate for present traffic built with Interstate or other public funds

MAJOR TOLL ROADS

Incorporated in the Interstate System

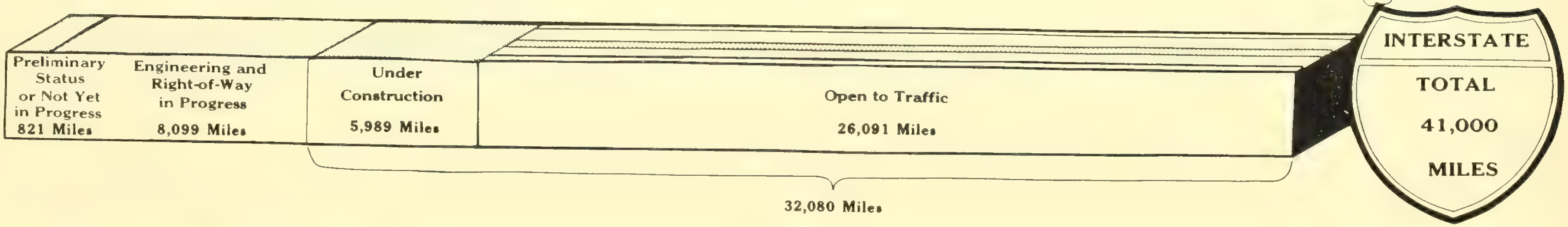
UNDER CONSTRUCTION

PRELIMINARY STATUS OR NOT YET IN PROGRESS

Plan preparation and right-of-way acquisition completed or underway on many portions of these sections

Scale of map does not permit showing of status in urban areas and for very short sections

U S DEPARTMENT OF TRANSPORTATION,
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

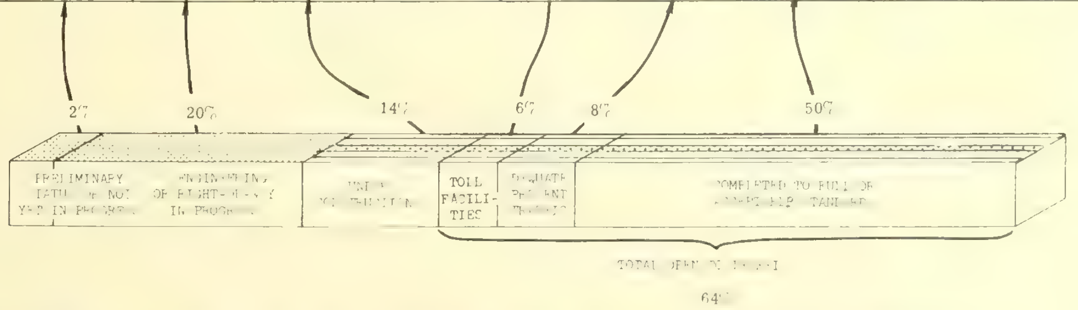


THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

IMPROVEMENT STATUS OF SYSTEM MILEAGE AS OF JUNE 30, 1968

TABLE I

STATE	PRELIMINARY STATUS OR NOT YET IN PROGRESS 1	WORK IN PROGRESS: ENGINEERING OR RIGHT-OF-WAY	WORK IN PROGRESS: UNDER CONSTRUCTION	WORK IN PROGRESS: TOTAL UNDERWAY	OPEN TO TRAFFIC: TOLL FACILITIES	OPEN TO TRAFFIC: IMPROVED TO STANDARDS ADEQUATE FOR PRESENT TRAFFIC	OPEN TO TRAFFIC: COMPLETED TO FULL OR ACCEPTABLE STANDARDS	OPEN TO TRAFFIC: TOTAL OPEN TO TRAFFIC	TOTAL DESIGNATED SYSTEM MILEAGE	STATE
ALABAMA	-	[illegible]	[illegible]	392.6	-	141.1	343.7	484.8	877.4	ALABAMA
ARIZONA	1.0	179.9	246.9	[illegible]	-	[illegible]	481.4	739.5	[illegible]	ARIZONA
ARKANSAS	-	57.5	126.9	[illegible]	-	4.3	[illegible]	334.5	518.9	ARKANSAS
CALIFORNIA	-	[illegible]	352.5	779.5	10.2	[illegible]	1,067.5	[illegible]	[illegible] 2/	CALIFORNIA
COLORADO	119.1	[illegible]	[illegible]	202.7	-	115.8	497.1	612.9	934.7 3/	COLORADO
CONNECTICUT	-	23.1	[illegible]	[illegible]	[illegible]	[illegible]	197.5	261.3	295.6	CONNECTICUT
DELAWARE	-	[illegible]	10.4	19.8	[illegible]	0.9	[illegible]	20.8	40.6	DELAWARE
FLORIDA	[illegible]	328.3	124.5	[illegible]	44.8	-	[illegible]	689.5	[illegible]	FLORIDA
GEORGIA	-	307.2	217.6	[illegible]	-	7.0	[illegible]	583.6	1,108.4	GEORGIA
HAWAII	[illegible]	25.0	3.8	28.8	-	1.6	9.9	11.5	51.9	HAWAII
IDAHO	-	128.9	87.6	[illegible]	-	102.6	[illegible]	391.8	608.3	IDAHO
ILLINOIS	[illegible]	370.3	242.3	[illegible]	156.0	143.0	691.9	[illegible]	[illegible]	ILLINOIS
INDIANA	-	[illegible]	230.9	438.7	156.9	[illegible]	504.1	[illegible]	[illegible]	INDIANA
IOWA	-	[illegible]	[illegible]	206.3	[illegible]	-	499.1	502.7	709.0	IOWA
KANSAS	0.1	101.2	[illegible]	[illegible]	185.9	0.3	439.8	626.0	800.9	KANSAS
KENTUCKY	-	[illegible]	168.9	[illegible]	39.2	[illegible]	372.9	[illegible]	[illegible]	KENTUCKY
LOUISIANA	-	202.0	[illegible]	[illegible]	-	1.8	283.2	285.0	[illegible]	LOUISIANA
MAINE	1.8	[illegible]	1.2	[illegible]	58.0	99.4	[illegible]	275.7	312.1	MAINE
MARYLAND	[illegible]	26.7	[illegible]	58.5	[illegible]	70.9	[illegible]	[illegible]	[illegible]	MARYLAND
MASSACHUSETTS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	196.0	359.2	451.1	MASSACHUSETTS
MICHIGAN	-	166.1	[illegible]	230.7	[illegible]	[illegible]	[illegible]	850.5	1,081.2	MICHIGAN
MINNESOTA	-	280.9	218.6	[illegible]	-	[illegible]	362.2	404.5	904.0	MINNESOTA
MISSISSIPPI	-	125.6	[illegible]	281.0	-	19.2	378.1	397.3	[illegible]	MISSISSIPPI
MISSOURI	[illegible]	[illegible]	[illegible]	333.1	[illegible]	[illegible]	611.4	[illegible]	1,119.9	MISSOURI
MONTANA	[illegible]	[illegible]	83.1	[illegible]	-	300.4	264.4	[illegible]	[illegible]	MONTANA
NEBRASKA	-	92.6	50.6	143.2	0.2	12.9	321.3	[illegible]	477.6	NEBRASKA
NEVADA	-	129.5	36.6	166.1	-	5.3	363.2	368.5	[illegible]	NEVADA
NEW HAMPSHIRE	[illegible]	30.0	14.1	44.1	22.0	[illegible]	[illegible]	[illegible]	214.9	NEW HAMPSHIRE
NEW JERSEY	[illegible]	98.9	64.9	163.8	46.3	[illegible]	89.2	[illegible]	[illegible]	NEW JERSEY
NEW MEXICO	[illegible]	[illegible]	118.5	322.7	-	61.0	577.2	[illegible]	998.4	NEW MEXICO
NEW YORK	[illegible]	[illegible]	90.2	[illegible]	[illegible]	[illegible]	504.0	1,048.0	[illegible]	NEW YORK
NORTH CAROLINA	-	200.0	131.9	[illegible]	-	17.3	[illegible]	[illegible]	770.2	NORTH CAROLINA
NORTH DAKOTA	[illegible]	47.2	68.8	116.0	-	[illegible]	340.3	392.2	570.8	NORTH DAKOTA
OHIO	[illegible]	190.9	[illegible]	[illegible]	[illegible]	55.0	829.3	1,090.7	1,530.6	OHIO
OKLAHOMA	-	58.5	146.8	205.3	[illegible]	23.3	394.7	[illegible]	797.4	OKLAHOMA
OREGON	[illegible]	[illegible]	[illegible]	[illegible]	-	111.1	537.8	648.9	[illegible]	OREGON
PENNSYLVANIA	[illegible]	171.8	342.1	513.9	[illegible]	[illegible]	656.0	[illegible]	1,575.7	PENNSYLVANIA
RHODE ISLAND	-	11.2	18.1	29.3	-	8.7	[illegible]	41.5	[illegible]	RHODE ISLAND
SOUTH CAROLINA	-	92.2	[illegible]	289.8	-	[illegible]	374.5	[illegible]	682.1	SOUTH CAROLINA
SOUTH DAKOTA	-	176.4	87.7	[illegible]	-	77.5	337.6	415.1	[illegible]	SOUTH DAKOTA
TENNESSEE	-	313.2	142.5	455.7	-	92.6	502.3	[illegible]	1,050.6	TENNESSEE
TEXAS	[illegible]	589.9	[illegible]	1,000.1	-	302.8	1,701.8	[illegible]	3,028.0	TEXAS
UTAH	50.8	[illegible]	171.0	[illegible]	-	[illegible]	273.3	309.6	933.8	UTAH
VERMONT	-	116.2	59.5	[illegible]	-	[illegible]	[illegible]	144.7	320.4	VERMONT
VIRGINIA	[illegible]	237.6	[illegible]	[illegible]	[illegible]	47.2	568.8	[illegible]	1,060.2	VIRGINIA
WASHINGTON	[illegible]	110.8	[illegible]	189.0	-	196.0	276.9	[illegible]	726.7	WASHINGTON
WEST VIRGINIA	45.7	166.3	69.1	235.4	87.2	0.3	145.8	[illegible]	[illegible]	WEST VIRGINIA
WISCONSIN	0.7	1.7	69.3	71.0	-	24.7	361.9	386.6	458.3	WISCONSIN
WYOMING	106.5	69.3	131.6	200.9	-	49.3	552.1	[illegible]	908.8	WYOMING
DISTRICT OF COLUMBIA	[illegible]	[illegible]	[illegible]	9.9	-	[illegible]	6.9	9.8	29.6	DISTRICT OF COLUMBIA
PENDING	[illegible] 1/								36.4 5/	PENDING
TOTAL	[illegible]	8,098.6	[illegible]	14,087.5	2,305.0	[illegible]	[illegible]	[illegible]	[illegible]	TOTAL



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1 Public hearings have been held on route location, and location studies are underway on many portions of the mileage in this column.

2/ Exclude the 17.2 mile Century Freeway (I-105) which was added to the system under the "Howard Bill."

3/ Excludes a 10.8 mile increase resulting from a major relocation of I-70, approved July 18, 1968. This increase will reduce the miles pending to 25.6 miles.

4/ Excludes the [illegible] mile [illegible] (I-195) which was added to the system under the "Howard Bill" but includes that portion of I-278 mileage (7.0) deleted under the same bill.

5/ Consists of mileage which has not been assigned to any specific route and is a reserve for final measurement of the system.

RESEARCH BRIEF

Some Steam Baths Are Not So Hot!—

Leonard Bean, Materials Division, Bureau of Public Roads [1]

In many procedures described for inorganic analysis, a steam bath is specified for certain digestions. For example, in American Society for Testing and Materials (ASTM) Designation C 114–67, Section 80.1, silica is separated from portland cement by digesting a mixture of the cement sample with NH_4Cl and HCl on a steam bath for 30 minutes.

In recent work at the Public Roads laboratory, it was made clear that merely specifying the use of a steam bath is not sufficient to ensure complete decomposition of the cement sample and separation of the silica and, thus, secure correct results for SiO_2, Al_2O_3, CaO, and MgO.

The Public Roads laboratory was relocated recently, and after installation of facilities at the new location, an insufficient supply of steam was furnished to heat the steam baths. Usually, low results were obtained for SiO_2—as much as 0.5 percent absolute—and high results for Al_2O_3, CaO and MgO. Under the conditions at the newly installed laboratory, 100 ml. of water in a covered 150-ml. beaker on a steam bath could be heated no higher than about 63° C. This source of heat was not sufficient to decompose a sample of cement properly. By modifying the supply of steam, sufficient heat was supplied to the water in the steam bath to maintain it at a rolling boil. Under these conditions, 100 ml. of water in a covered 150-ml. beaker could be kept at 83° to 90° C. Decomposition and dehydration o cement by the method cited then gav reasonable values for SiO_2 in standard sample of cement issued by the National Bureau o Standards, as well as reasonable values fo Al_2O_3, CaO, and MgO.

Accordingly, analytical chemists who use a steam bath for such separations should b certain that the heat source (steam, gas, or electricity) can maintain the water in the bath at a rolling boil.

Furthermore, it is recommended that bodies who write specifications for such analyses—ASTM, Federal Committees, American Association of State Highway Officials, etc. —tak cognizance of this possible source of error. A footnote or warning might well be inserted in procedures requiring the use of a steam bath for decompositions such as the one cited here.

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[1] Essentially, this same information appeared in a Letter To The Editor, Materials Research and Standards, vol. 8, No. 6, June 1968, p. 51, published by the ASTM.

Ultrasonic Instrument for Determining Local Scour at Bridge Piers

(*Continued from p. 96*)

single plane, and by controlling the transducer position through mechanical linkage, the complexity and cost of the instrument could be substantially reduced. If the mechanical unit were simplified, the electronics package could also be reduced. A single 12-volt rechargeable power source could be used and the display/test module eliminated or replaced by a small strip chart recorder. In addition, for applications not requiring a high degree of bottom definition, a smaller transducer could be used.

Either the basic research unit or a simplified version could be equipped for remote monitoring operations.

ACKNOWLEDGMENTS

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U.S. GOVERNMENT PRINTING OFFICE: 1968

PUBLICATIONS of the Bureau of Public Roads

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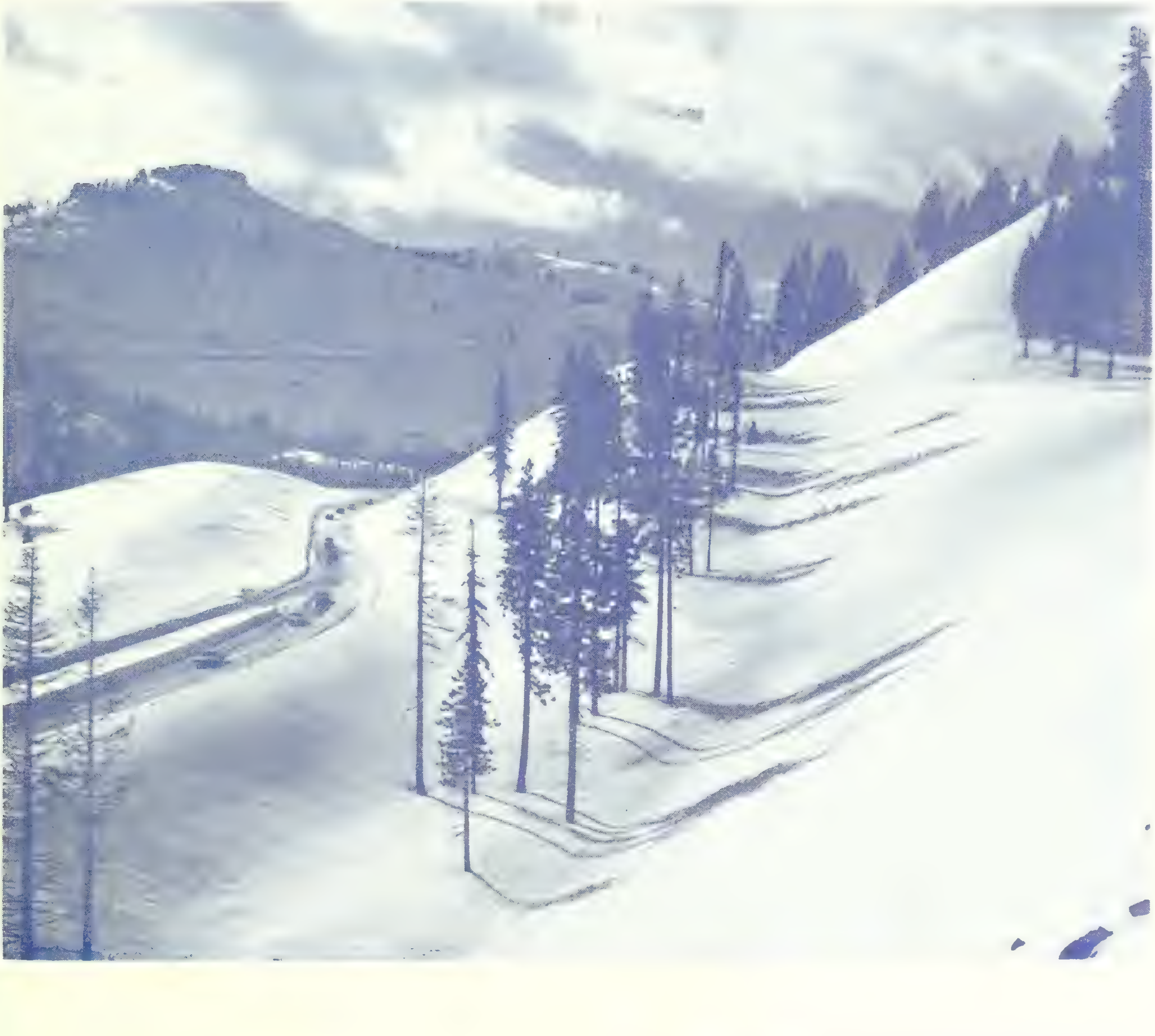

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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FEDERAL HIGHWAY ADMINISTRATION
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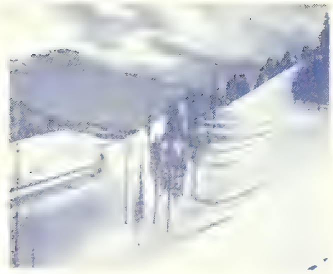

COVER

Eastbound lanes of Interstate Highway 80 near Donner Lake, Calif. Westbound lanes are near the right edge of the photograph.

THE BUREAU OF PUBLIC ROADS
FEDERAL HIGHWAY ADMINISTRATION
U.S. DEPARTMENT OF TRANSPORTATION
Washington, D.C. 20591

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Trailer-on-flatcar (TOFC) is an expanding form of freight service that combines the established efficiencies of different modes of transportation.

Intermodal Freight Transportation in the United States

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by EARLE S. NEWMAN,
Economist, Economics and
Requirements Division

Intermodal freight transportation is dealt with in this article as it is in the United States today and as it is expected to evolve in the next decade or two. Rather than provide only a single-mode service, the carriers, through increased coordination, now offer the public an improved intermodal service, the inherent advantages of which are a strong impetus to further its growth. Competitive modes of transportation are benefiting from this coordinated service by capturing the best features of each mode. Containerization, creating a new role for motor and rail carriers, has enabled them to develop a more convenient, faster, and versatile service. New cost-saving efficiencies will assure the continued growth of trailer-on-flatcar service, and may produce a surge of traffic in its second or third decade of operation. What eventually is best for the shipper will govern the rate of intermodal-transportation growth. Continued close relations among competitive regulated carriers and shippers may reverse the trend to operate privately owned equipment. Trailer-on-flatcar service has the potential to increase the efficiency of basic transport resources and reduce accidents, fatalities, and injuries on the highways.

Introduction

IN recent years, coordination between highway and railroad transportation has progressed notably. Since it was revived in 1954, Trailer-on-flatcar (TOFC) service has probably expanded faster than any other transportation service. Should TOFC service continue to expand markedly, it will affect the use of the Interstate highway system by altering the role of highway carriers in the transportation of long-haul intercity freight.

The need for more coordination among different segments of the transport industry has been apparent for a long time. With the passage of the Hepburn Act in 1906, Congress recognized the importance of coordination among carriers. Under the provisions of the Act, carriers were to establish through-routes and joint rates. The Transportation Act of 1920 also provided for coordination in the use of transportation support facilities.

The need for coordinated transportation service is also evident in the national transportation policy which states that the Interstate Commerce Act is to bring about "... development, coordinating, and preserving a national transportation system by water, highway and rail, as well as other means ..." In his message on *The Transportation System of Our Nation,* submitted to Congress on April 5, 1962, the President stated "... we must now consider the Nation's transportation network as an articulated and closely linked system rather than an uncoordinated set of independent entities." The Presidential Message submitted to Congress in March 1966 again emphasized the need for a "... coordinated transportation system that permits travellers and goods to move conveniently

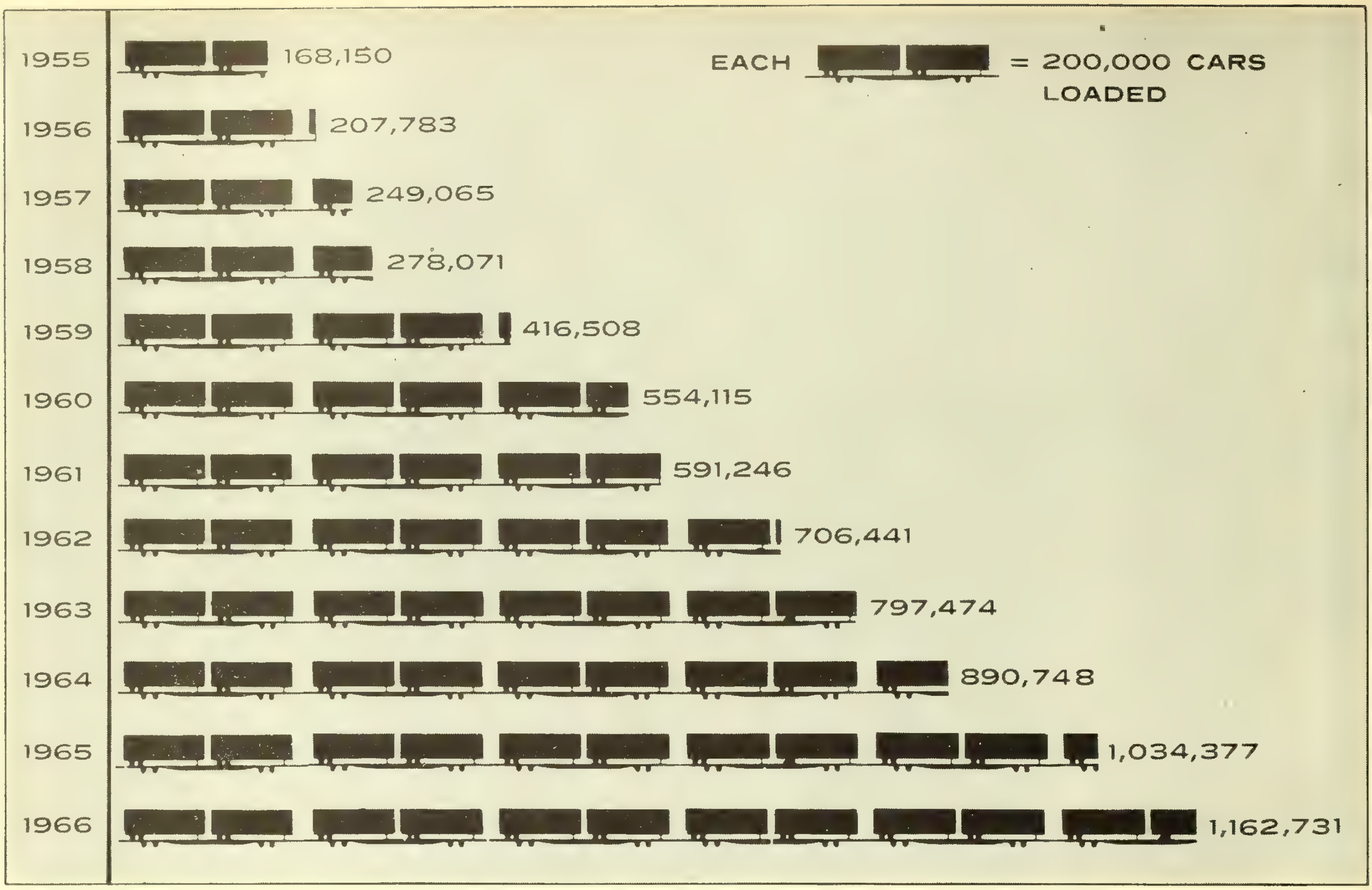

Figure 1.—**Piggyback *revenue carloadings, 1955–66.***

and efficiently from one means of transportation to another, using the best characteristics of each."

The total cost of transportation for the Nation is approximately 20 percent of the Gross National Product (GNP) or about $150 billion annually (*1*).[1] Approximately half of the expense for freight transportation may go toward paying for what one member of the transportation industry calls the "grand tiddly-winks game of shuffling goods on docks, platforms, between vehicles, and in other side expenses like packaging, damage claims, insurance and the like".(*2*)

Regardless of the fact that coordination between modes of transportation has been held to be essential, there was very little intermodal coordination until 1954, when TOFC service gained momentum. This service has experienced continuous growth since that time, proving beneficial to both carriers and shippers.

The purpose of this article is to consider possible shifts in intercity freight traffic to TOFC service by assessing the relation of this service to the land transport system and shippers and determining its present status and potential growth.

Progress of TOFC Service

It is generally recognized that the growth of TOFC service did not begin until 1954. Since that time, the annual revenue carloadings have increased from 168,150 in 1955 to approximately 1,207,000 in 1967, representing a 618-percent increase, as shown in table 1. Since 1955, annual carloadings have increased each year over the preceding year, as reflected in figure 1.

Although the law has long permitted joint service by various types of carriers, TOFC did not develop as a major transportation service until a variety of arrangements that met the needs and requirements of the shipping public were available. Accordingly, the future growth of this service will probably be contingent on continual flexibility of service and variety of pricing systems.

The Interstate Commerce Commission (ICC) fostered the initial growth of TOFC service by permitting it to develop over a 10-year period, giving it sufficient time for the principal issues to crystalize, before conducting its investigation and proposing rules. In 1954, the ICC conducted an investigation in the so-called New Haven case (293 I.C.C. 93) in which certain practices had been slowly developing in TOFC service. The ICC's approval of these practices resulted in a strong renewal of interest in coordinated transportation by the common carriers and in participation of additional carriers in specific plans.

With the steady growth of TOFC service, problems have arisen; some have been resolved and others will require long and deliberate consideration (*3*). Problems still exist in the terminal areas, standardization of equipment, stabilization of rates, and utilization of equipment. However, to cope with problems that confront the railroad industry, a National Railroad Piggyback Association (NRPA) has been organized.

The bylaws of NRPA state that its aims are to "foster, protect and promote the interests of railroads engaged in the business of handling traffic by piggyback, to advance such interests throughout the United States and elsewhere through cooperation and organization, to stimulate the widespread use and recognition of such railroads and to assist in the solution of problems affecting the membership." The NRPA will deal with significant sales, operations, terminals, containerization, car equipment, and trailers. It is too early to appraise NRPA's contribution to the solution of problems or to assess what future part this organization will play to promote intermodal freight transportation. The organization is not intermodal in its membership, nor does it have any enforcement powers among its members.

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[1] Italic numbers in parentheses identify the references listed on p. 114.

Developments in TOFC service thus far have left little doubt that there are opportunities for additional growth as a result of the advantages that exist in coordinated transportation. In the ICC's investigation Ex Parte 230, of TOFC service, the examiners described *piggyback* as "one of the most dynamic formulas for transportation this country has ever seen." They also stated that "the bounds of this service and its total effect on transportation are as wide and long as the imagination of the men who are providing this service to the Nation."

At this time however, when much effort is being made to stabilize practices into fixed patterns of operation, the potential of TOFC service is difficult to assess because of frequently occurring internal and external changes.

Piggyback service may have a tremendous potential for both motor and rail common carrier segments of the transportation industry. It could be the motivating factor in reversing the following predominant trends:

- The use of privately-owned transportation equipment instead of public carriers.
- The lack of cooperation among highly competitive common-carrier modes of transportation.
- Increasing private-carrier transportation costs.

Major Categories of Traffic Adaptable to TOFC Service

During the period following World War II, the trend in the use of private transportation in lieu of public transportation has been substantial. The common carrier industry, both rail and motor, continues each year to lose an appreciable volume of intercity freight traffic to the unregulated private operators. The Nation's estimated freight bill for intercity domestic surface transportation in 1966 was approximately $39 billion. This revenue was distributed among the different modes of transportation as follows:

Mode:	*Amount (billions of dollars)*
Motor—unregulated, intercity	17. 34
Motor—ICC regulated	10. 15
Railroads	10. 92
Railway Express	. 40
Bus	. 08

Approximately 44 percent of the freight bill was for unregulated highway transportation.

Some of the inherent advantages in *piggyback* service, such as reduction in damage, elimination of pilferage, reduction in labor requirements, greater utilization of carrier equipment, lower freight costs, and contribution to reduction of highway accidents, should substantially help reverse the trend to use private equipment.

During the past three decades, the motor carrier industry has continually grown, and it has been responsible for generating much new traffic and for diverting traffic from competitive modes of transportation. Motor carriers have demonstrated an ability to meet the entire transportation requirements of certain industries. Industry now considers proximity to good highways a major factor in selecting plant locations (*4*).

Table 1.—Increase in revenue carloadings of trailer-on-flat-car traffic—class I railroads, 1955–67

Year	Number[1]	Increase		
		Over preceding year	Over preceding year	Over 1955
	Carloads	*Carloads*	*Percent*	*Percent*
1955	168, 150	--------	--------	------
1956	207, 783	39, 633	23. 6	23. 6
1957	249, 065	41, 282	19. 9	48. 1
1958	278, 071	29, 006	11. 6	65. 4
1959	416, 508	138, 437	49. 8	147. 7
1960	554, 115	137, 607	33. 0	229. 5
1961	591, 246	37, 131	6. 7	251. 6
1962	706, 441	115, 195	19. 5	320. 1
1963	797, 474	91, 033	12. 9	374. 3
1964	890, 216	92, 742	11. 6	429. 4
1965	1, 034, 377	144, 161	16. 2	515. 2
1966	1, 162, 731	128, 354	12. 4	591. 5
1967	1, 207, 242	44, 511	3. 8	618. 0

[1] Source: Association of American Railroads, Form CS 54A.

Because *piggyback* service offers many potential advantages to both modes of transportation, a much higher degree of coordination between railroads and motor carriers should result. The burden is on the regulated carriers—both motor and rail—to further develop programs capable of meeting private carrier competition. TOFC services offer the greatest opportunity to accomplish this.

Many freight movements require no separate containers, such as volume shipments of bulk commodities—coal, iron ore, grain and liquids—for which the walls of the vehicles serve as the containers. Other examples of commodities requiring little, if any, containerization are new automobiles, structural steel, and building stone.

Although commodities that do not have to be containerized make up a large portion of the total tonnage transported in intercity freight service, they do not represent as large a portion of the total cost paid for transportation. Package goods, small shipments, and volume shipments of manufactured items, which are readily adaptable to TOFC service, are far more costly to handle, and therefore offer larger potential savings in handling and transportation costs.

National Transportation Policy Report No. 445, prepared in 1961 by the Committee on Commerce, U.S. Senate, stated that approximately three-fourths of the intercity Interstate freight carriage is by regulated carrier. However, much of the post-war expansion in transportation has accrued to the benefit of the unregulated carriers. It was estimated that 25 percent of the freight moving over the highway was moving in illegal service. It was predicted that by 1975, only 61 percent of the intercity freight will be handled by regulated carriers, and that the principal impact will be on the railroads. The report projected that the regulated carriers will be handling 1,111 billion ton-miles and the unregulated carriers 718 billion ton-miles of freight in intercity service.

The requirement for intercity freight transportation is usually related to the output of goods and construction, which represents about two-thirds of the GNP. Based on this relationship, United Research, Incorporated, Cambridge, Mass., made the projections of intercity freight ton-miles through1985, shown in table 2.

The year 1962 marked the first post-war year that the collective intercity traffic (total ton-miles) of regulated carriers (rail, highway, and water) gained on their unregulated competitors. Compared with 1962, the relative percentage of distribution remained about the same in 1963 and 1964; there was a slight change in favor of the regulated carriers in 1965 and 1966.

The best opportunity for motor and rail common carriers to generate additional volumes of remunerative traffic is to offer a sufficiently attractive coordinated service to divert freight from the exempt and private carriers. Outside the bulk commodity area there is a great deal of high-revenue-producing traffic currently being handled in unregulated transportation which lends itself to TOFC service. It is possible that much of this freight is being handled in equipment loaded in one direction at an unreasonable cost to the shipper.

Table 2.—Projection of intercity freight

Year	Value of goods and construction	Transportation required	
		Per dollar of goods and construction	Total
	Billions of dollars	*Ton-miles*	*Billions of ton-miles*
1970	457	3. 8	1. 737
1975	500	3. 7	2. 012
1980	631	3. 6	2. 271
1985	754	3. 5	2. 639

Source: *Future U.S. Transportation Needs* by A. H. Norling, United Research, Inc., Cambridge, Mass., 1963, p. VIII–4.

During the past 25 years, the unregulated transportation industry has grown tremendously. Operating independently, neither the motor- nor rail-regulated carriers seems to be able to cope with this competition to the extent that either mode can any longer control the majority of the intercity tonnage or revenue. The traffic handled in unregulated service represents the largest market of potential remunerative traffic adaptable to TOFC service. There has been a relative decline in railroad use, as a factor in surface transportation, and a noticeable inability of the regulated motor carriers alone to significantly reverse the trend in traffic handled by unregulated motor carriers. To make substantial inroads on unregulated competition, and capture or recover a significant part of the intercity freight from the unregulated carriers, the regulated carriers apparently will have to participate to a greater extent in coordinated TOFC service.

It is the freight that is usually the most costly to transport that justifies the inaugura-

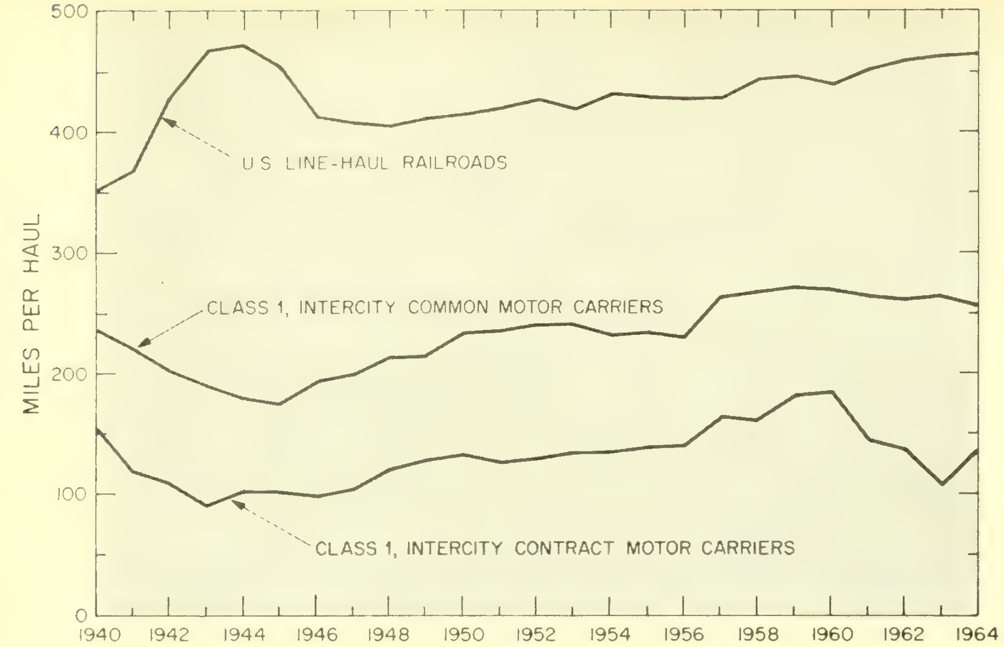

Figure 2.—Length of haul by truck and rail, 1940–64.[2]

tion and operation of privately-owned motor service. Because it consists of high-revenue-producing traffic and represents about 40 to 45 percent of the intercity freight, this traffic should provide a strong impetus to regulated motor and rail carriers to expand and perfect their coordinated TOFC services to recapture a larger portion of this market.

Although the motor carriers have grown rapidly because of their important part in the economic development of the United States, about half of their ton-mileage consists of shipments of more than 400 miles, which is subject to being handled in TOFC service. In figure 2 is shown the length of haul by class I intercity motor carriers (common and contract) and all U.S. line-haul railroads, for the years 1940–64. Between 1940 and 1964, the length of haul for the class I common motor carriers increased 9 percent, while the railroads increased 33 percent and the contract motor carriers declined 10 percent. Comparable data are not readily available for private motor carriers, but it is reasonable to believe that during the same period, their average length of haul has increased substantially.

Service and Rates

The inauguration of TOFC service was the first time that two different modes of transportation, motor and rail, combined the best features of their respective services into a large-scale transport operation. This coordinated transportation offers considerable flexibility to shippers, an improvement in reliability of service over a single mode of transportation and, most important, simplified rate structures. The shipper has the combined advantage of door-to-door service by motor carriers, and economical long-haul service by the rail carriers.

In their production and distribution processess, industrial firms weigh the costs of inventory, warehousing, and handling to price their products on a competitive basis. Accordingly, any transit-time saving also saves money and can reduce the overall costs of manufacturing and marketing products. This affords the carriers an opportunity to pass these advantages on to consumers.

Another significant advantage of TOFC service is single-carrier control, whereby the shipper can obtain reports on the status of the shipment from the originating carrier. There has also been a substantial reduction in freight damage when goods are shipped by TOFC service. Although there is probably no measure of the extent of damage reduction, it is the consensus of both shippers and carriers that considerably less damage occurs than when goods are shipped in partial loads—less carload or less truckload service.

Faster service, fewer losses, less damage, and high utilization of equipment will substantially influence the quality of service and the rates in TOFC transportation. If TOFC service enables the motor carriers to keep their rising costs in check, and at the same time provides a margin of profit for the rail carriers, these two modes of common carrier transportation will recapture more of the present traffic handled over-the-road in private transportation; working together, they should be able to slacken the rate of growth and reverse the trend in the use of private transportation.

Before World War II, railroads enjoyed a distinct advantage in freight transportation. During the war a major impetus was given to the growth of the motor carrier industry. Following the war, these two modes of transportation competed for traffic, primarily on a service basis rather than on a cost basis. The greater flexibility of the motor carrier services enabled them to generate much new traffic and to divert an appreciable amount of the more remunerative traffic from the railroads. However, with the establishment and growth of TOFC service since World War II, emphasis has shifted from service competition to rate competition.

Prior to TOFC service, much of the litigation before the ICC involved the question of whether rates were too high; now the litigation usually involves the question of whether rates are too low. For the regulated carriers, this litigation has exerted a downward pressure on the rate structures, which, if continued, should result in attracting more traffic from the private carriers.

One of the most significant aspects of TOFC services is the economic implications of the new departures in rate-making that accompanied the inauguration of TOFC operations on a large scale. The rates provided under the TOFC plans are based on simple principles, compared to the conventional procedures and factors which previously entered into rate-making. Consequently, substantial advantages from both administrative and monetary standpoints are accruing to the participants in TOFC service.

Coordination Between Motor Carriers and Railroads

While some transportation coordination has been permitted under the provisions of the Interstate Commerce Act—through-route and joint-rate arrangements—generally, there was very little actual coordination in this country between rail and motor carriers prior to 1954, when *piggyback* service began.

The ICC has contended that additional authority is needed for the establishment of through routes and joint rates. Under the existing statutes, the ICC cannot compel motor common carriers of property to enter into joint-rate and through-route arrangements with each other, or with common carriers of other modes. This gap in authority has probably been one of the reasons why there has not been greater coordination among unlike modes of transportation. When in the public interest, however, the ICC has repeatedly requested authority from Congress to require through routes and joint rates between motor common carriers of property and between motor carriers and common carriers by rail, express, and water.

The development of TOFC service represents an appreciable step forward by the motor and rail carriers to reap the potential benefits of coordinated services. Thus far, coordination between motor carriers and railroad industries in TOFC service may be discussed briefly under four topics—managerial, marketing, technological, and operationas.

Managerial coordination

The further advancement of TOFC service will depend, to a considerable extent, on the degree of managerial coordination between the motor and rail carriers. Within the framework of the five TOFC plans exists the maximum opportunity for such coordination. A motor carrier, for example, may solicit the freight traffic, provide the documentation, labor, motor power for pickups, delivery of trailer, and collect the transportation charges, thereby depending on the railroad only to provide the power and railcar for the rail-haul portion of the movement. This represents a high degree of coordination between two unlike modes of transportation.

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[2] Source: *Transportation Statistics in the United States*, Bureau of Transport Economics and Statistics, Interstate Commerce Commission.

This coordination offers certain inherent advantages to carriers. Managers gain greater knowledge of the operating characteristics of other forms of transportation and become more familiar with the managerial and economic aspects of competitive modes of transportation. The continual growth of TOFC service during the past 10 years is due to this coordination, which has not only resulted in substantial advantages for both modes of transportation, but also has provided the public with improved service.

Marketing coordination in TOFC service

TOFC service offers considerable latitude for coordination in marketing a *packaged transportation service.* It represents a new era for the shipping public, as it affords an opportunity to consolidate the advantages of each mode of transportation, thereby making available the best service to suit the shipper's requirements at a reduction in cost for furnishing the transportation. The major customers do not rely exclusively—from a service and cost standpoint—on either mode of transportation. The services that the two modes render within the sphere of their respective operations provide transport capability, but single mode capability does not provide all the inherent advantages of both modes.

Technological coordination

The third area of coordination in TOFC service is the technological area. The potential of combining the technical features of each mode of transportation has been recognized for many years, but only recently have the carriers begun to take advantage of the opportunity. *Piggyback* focuses attention on the advantage of trucking flexibility to perform pickup and delivery of short-haul and medium-haul service, as well as on the cost advantage of rail in some medium-haul and most long-haul service.

It has been necessary for the motor and rail carriers to modify existing equipment and to develop new equipment, including the *piggyback* car and storage and loading facilities, to meet the technological requirements for an efficient service. Management of both modes of transportation has been receptive to a high degree of coordination in technological areas, as they have received the benefits of lower operating costs.

Operations coordination

Since 1954, a general atmosphere of coordination between motor and rail carriers has developed. It can be concluded that the shipper advocates the maximum amount of coordination between modes of transportation to provide direct, expeditious service with a minimum of administrative expense at the most economical rate.

To attract the traffic, each mode of transportation endeavors to render a superior service at sufficiently compensatory rates. Both the rail and motor carriers have certain inherent advantages to offer regarding service and rates. Much freight traffic is susceptible to movement by either mode of transportation, and it is within this area that the greatest opportunity for coordination between the modes exists. By unifying the advantages of each mode, the carriers are capable of offering a combination service heretofore not readily available to the public.

Through cooperative operations and development of the respective modes, the carriers are reaching new peaks of efficiency. Collectively, they are capable of rendering a superior service in many categories of long-haul traffic. However, the continual growth of private transportation has had a significant impact in promoting coordination between regulated motor and rail transportation. Generally, private transportation affects the most desirable traffic—the type that would produce the highest rate of revenue for the regulated carriers.

To illustrate the effect of private transportation, meat and dairy products are high-revenue producing traffic for both rail and motor carriers. In the 1963 Census of Transportation, the Commodity Transportation Survey on meat and dairy products revealed that about 43 million tons of this commodity were shipped beyond the local area. Private trucks handled more than half of the total tonnage shipped distances of less than 200 miles; motor carriers dominated the middle-distance range—from about 200 to 800 miles—and railroads transported more than half of total shipments over longer distances. Percentage distribution of this traffic among modes on a tonnage and ton-mile basis is shown by the following tabulation

Mode of transportation:	*Tons, percent*	*Ton-miles, percent*
Private	42	16
Motor	30	36
Rail	28	48

More of this long-haul traffic could have been handled by coordinated rail-motor service.

Once a shipper is committed to private transportation because of his heavy investment in equipment, he is not easily persuaded to abandon his carrier operations. Although it is not evident that any large shippers have discontinued their private transport operations because of TOFC service, many have refrained from increasing the size of their transport operations because of the availability of TOFC service.

The potential overall advantages that accrue to regulated motor and rail carriers should enable them to offer a coordinated service capable of prompting the operators of private carriage to rely, to a greater extent, on public transportation.

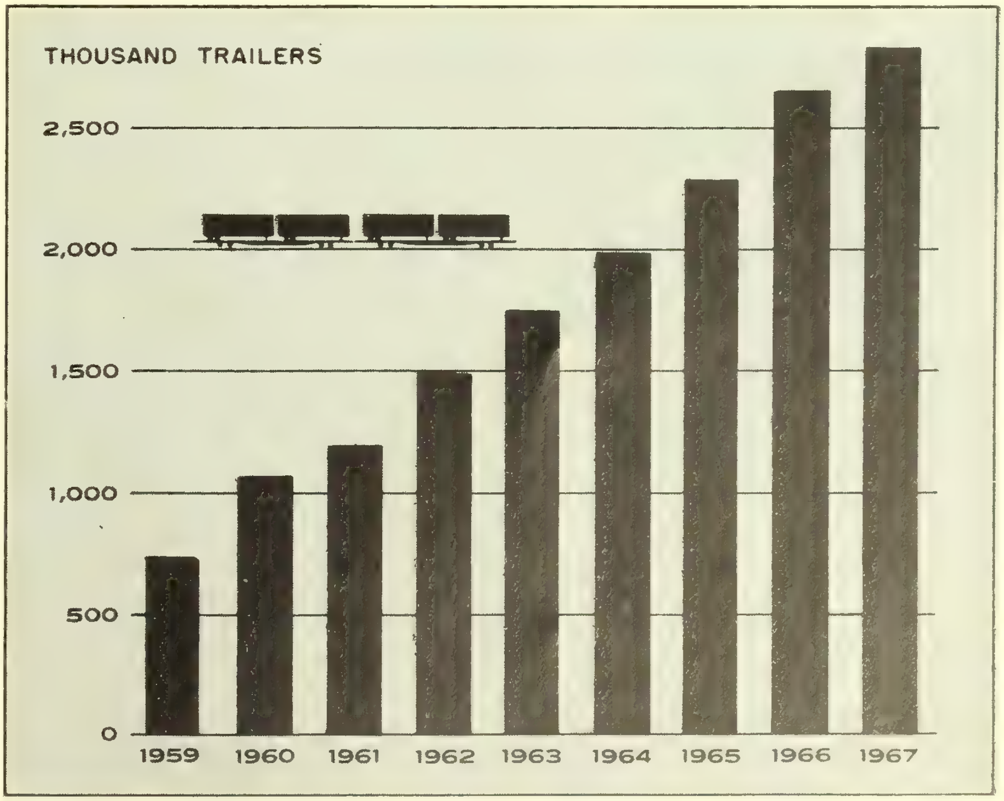

Figure 3.—**Piggyback** *growth, 1959–67—class 1 railroads.*

Table 3.—Projections of number of carloads and truckloads in trailer-on-flatcar revenue service, 1967–87

[Based on an annual increase of the percentages shown]

Year	Projected loads hauled					
	Annual increase—6 percent		Annual increase—8 percent		Annual increase—10 percent	
	Carloads	Truckloads [1]	Carloads	Truckloads [1]	Carloads	Truckloads [1]
	Number	*Number*	*Number*	*Number*	*Number*	*Number*
1967	1, 207, 242	2, 052, 311	1, 207, 242	2, 052, 311	1, 207, 242	2, 052, 311
1968	1, 279, 677	2, 175, 451	1, 303, 821	2, 216, 496	1, 327, 966	2, 257, 542
1969	1, 356, 458	2, 305, 979	1, 408, 127	2, 393, 816	1, 460, 763	2, 483, 297
1970	1, 437, 845	2, 444, 337	1, 520, 777	2, 585, 321	1, 606, 839	2, 731, 626
1971	1, 524, 116	2, 590, 997	1, 642, 439	2, 792, 146	1, 767, 532	3, 004, 789
1972	1, 615, 563	2, 746, 457	1, 773, 834	3, 015, 518	1, 944, 275	3, 305, 268
1973	1, 712, 497	2, 911, 245	1, 915, 741	3, 256, 760	2, 138, 703	3, 635, 795
1974	1, 815, 247	3, 085, 920	2, 069, 000	3, 517, 300	2, 352, 573	3, 999, 374
1975	1, 924, 162	3, 271, 075	2, 234, 520	3, 798, 684	2, 587, 830	4, 399, 311
1976	2, 039, 612	3, 467, 340	2, 413, 282	4, 102, 579	2, 846, 613	4, 839, 242
1977	2, 161, 989	3, 675, 381	2, 606, 345	4, 430, 787	3, 131, 274	5, 323, 166
1978	2, 291, 708	3, 895, 904	2, 814, 853	4, 785, 250	3, 444, 401	5, 855, 482
1979	2, 429, 210	4, 129, 657	3, 040, 041	5, 168, 070	3, 788, 841	6, 441, 030
1980	2, 574, 963	4, 377, 437	3, 283, 244	5, 581, 515	4, 167, 725	7, 085, 133
1981	2, 729, 461	4, 640, 084	3, 545, 904	6, 028, 037	4, 584, 498	7, 793, 647
1982	2, 893, 229	4, 918, 489	3, 829, 576	6, 510, 279	5, 042, 948	8, 573, 012
1983	3, 066, 823	5, 213, 599	4, 135, 942	7, 031, 101	5, 547, 243	9, 430, 313
1984	3, 250, 832	5, 526, 414	4, 466, 817	7, 593, 589	6, 101, 967	10, 373, 344
1985	3, 445, 882	5, 857, 999	4, 824, 162	8, 201, 075	6, 712, 164	11, 410, 679
1986	3, 652, 635	6, 209, 480	5, 210, 095	8, 857, 162	7, 383, 380	12, 551, 746

[1] Based on an average of 1.7 trailers per carload.

Growth of TOFC Service

As recorded in table 1, the revenue carloadings handled by TOFC service since 1955 have continually increased. The approximately 1,200,000 carloads of *piggyback* freight, transported in 1967, represent approximately 2 million loaded truck-trailers. Additionally, more than 800,000 empty trailers were handled for a total of about 2,800,000 trailers moved in rail service. The carriage of trailers in rail service increased from approximately 740,000 in 1959, to 2,800,000 in 1967, approximately a 400 percent increase.

TOFC service expanded about 24 percent between 1955 and 1956. (See table 1.) In 1959, it reached an annual growth peak of approximately 50 percent over the preceding year. There have been variations in the annual percentage increases over the preceding years, from a low of 3.8 percent in 1967 to a high of 49.8 percent in 1959. The large percentage gain between 1958 and 1959 was caused by more carriers offering TOFC service and the increased total volume of freight traffic.

During the last five years, TOFC service has grown an average of approximately 11 percent annually. The trend of *piggyback* service—even though fluctuating in rate of growth—is still in a stage of considerable expansion. (See fig. 3.) In each of the past 12 years, except 1961 and 1967, growth has exceeded 11 percent (table 1).

A conceptual approach to the growth of TOFC service has been developed showing increases in carloads and truckloads, based on an annual growth of 6, 8, and 10 percent respectively, predicated on an average of 1.7 trailers per flatcar (table 3). The validity of these projections should be accepted in terms of the principal factors directly affecting TOFC service. The ICC rules governing TOFC service, enactment of the Trade Simplification Act and the size and weight legislation, are significant factors that will affect the future growth rate of this service.

It is estimated that at least 6 million and possibly 12 million trailers will be transported in TOFC service by 1986—a three to sixfold increase. This is a conservative projection, as it represents a range of from considerably less than the fivefold increase of the past 10 years to a maximum of a sixfold increase in the next 18 years.

Once the pattern of TOFC operations is established under the factors identified above, it will undergo a sustained period of stable expansion.

Conclusions

Today, the competitive situation in intermodal transportation is intensified. Since World War II, improved highways and technological progress have resulted in longer and heavier truck hauls. The highway trailer has grown to the point of having the appearance and nearly the capacity of a boxcar. Industry is no longer attached exclusively to the rail heads. However, TOFC is a modern method of transferring containers and providing industry, regardless of its location, with adequate, efficient, and economical transportation, the further growth of which is now assured.

The past 10-year period has brought forth sufficient developments in TOFC service to support some conclusions and forecasts regarding its continued growth for the future. Although TOFC service, in a sense, may still be a transportation infant, it has considerable potential for substantial additional growth. Further experience will provide more improvements in service, pricing patterns, and coordination between competitors.

Piggyback is definitely a breakthrough in the barrier of cooperation that existed between the regulated railroads and motor carriers, both of which have a common, influential competitor in the unregulated carriers.

During this transitional period, the available statistical data on TOFC operations and services throughout the Nation is not sufficiently refined to reveal the precise answer to many questions such as: (1) To what extent has *piggybacking* meant the actual diversion of traffic from competitive modes of transportation? (2) Is traffic being handled at noncompensatory rates? (3) How much revenue is derived by the respective modes from TOFC traffic? and (4) What long-range impact will this rapidly developing system of coordinated transportation have on the Federal-aid highway system?

Thus far, there is no sign of total TOFC traffic leveling off. But neither have all the major factors that will influence its development settled into a sufficiently definite pattern to permit forecasting its future rate of growth, other than in the general manner shown here.

A true indication of the initial impact of TOFC service on highway freight is not yet in sight. It is reasonable to conclude that there will be considerably more coordinated service, and in the reasonably near future it should be possible to measure the impact of TOFC service on highway freight and its potential effect on the highway system.

REFERENCES

(*1*) *Transportation Facts and Trends,* Transportation Association of America, fifth edition, Washington, D.C., April 1968.

(*2*) *Containers: The Right Track?* Railway Age, vol. 148, No. 22, May 30, 1960, p. 9.

(*3*) *Piggyback—Progress Has Its Problems,* Railway Age, vol. 158, No. 22, May 31, 1965, pp. 54–66.

(*4*) *Highways, Trucks, and New Industry,* by Dept. of Research and Transport Economics, American Trucking Association, Inc., Washington, D.C., May 1963.

When highway trailers are hauled on rail flatcars in piggyback *operations, highway transportation is an essential part of the overall service.*

Highways and Rail Piggybacking

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by[1] ALEXANDER FRENCH,
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Although the trend to transport freight by hauling highway trailers on flatcars, piggybacking, *has been steadily upward, this intermodal freight service apparently has not retarded the growth of highway freight transportation. As pointed out in this article, it is unlikely that piggyback will replace highway transportation for moving freight over short distances; it is more likely to eliminate a substantial amount of intercity highway freight movement over long distances. Even though a large part of the long-distance highway freight travel would be converted to* piggyback *transportation, short distance highway travel should increase to handle movement to and from* piggyback *terminals. Highway planners must be attentive to the need for a high level-of-service on the arterial highway networks providing access between* piggyback *terminals and the origins and destinations of shipments.*

Introduction

THE transportation of highway semitrailers and interchangeable highway rail containers by rail flatcar provides efficient long-distance transportation of commodities. In the study reported here, this intermodal form of freight transportation, called *piggyback*,[2] was analyzed to determine whether a substantial shift in the highway share of total intercity ton-miles or other significant effects on highway administration, planning, or design are likely. It was concluded that *piggybacking* did not significantly dampen the growth of highway freight transportation during the past 10 years, nor is it likely to have a retarding effect in the future.

Special planning and design analyses are recommended for highways serving *piggyback* terminals to assure efficient intermodal operation. Special, highly localized problems could be caused by the concentration of semitrailer combinations on highways serving *piggyback* terminals during the periods immediately before and after loading and unloading of trailer trains. These problems can be readily identified and analyzed as part of the highway planning process. Roadways can then be designed to provide the necessary capacity for traffic flow and, where necessary, pavements affected by a high frequency of heavy axles can be strengthened.

National Trends

Trends in intercity rail and highway cargo transportation since 1945, based on a 1961 index, are shown in figure 1. Data on *piggyback* ton-miles can be estimated, beginning

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[1] Assisting in collecting and analyzing the material were T. S. Dickerson and W. J. Page, Highway Research Engineers; J. F. Petring, Highway Engineer Trainee; and A. J. Simms, Statistical Assistant.

[2] *Piggyback* refers to the transportation of highway trailers on railroad flatcars (TOFC) and to the transportation of interchangeable containers—similar to semitrailer van bodies—that are designed for transportation by railroad and highway semitrailer (COFC).

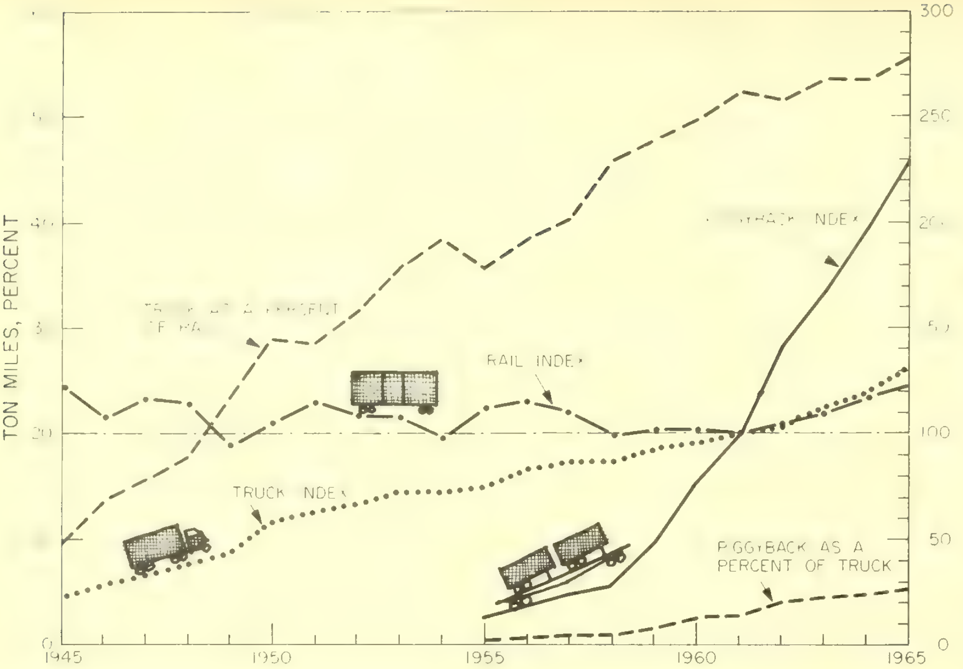

Figure 1.—Trends in intercity cargo transportation by piggyback, rail, and highway.

in 1955, as shown in table 1. From the index lines in figure 1, it is evident that since World War II, rail traffic has remained nearly constant, while truck and *piggyback* have increased—truck traffic increasing steadily and *piggyback* at a much higher rate. The line representing truck ton-miles as a percent of rail, when compared with the line for *piggyback* ton-miles as a percent of truck, indicates that while *piggyback* is growing at a very rapid rate, truck ton-mileage continues to grow faster with respect to rail than does *piggyback* with respect to truck.

Trends in relationships of intercity ton-miles hauled by rail, highway, and *piggyback* are shown in table 1. The share hauled by rail and *piggyback* together has declined from nearly 75 percent in 1955 to about 65 percent in 1965, while the highway share has increased from about 26 percent to 35 percent in the same period. The annual increase for total ton-miles and ton-miles by each of the three categories is also shown in the table. *Piggyback* has been gaining an increasing share of the annual increase of land vehicular ton-miles. Although *piggyback* operations have apparently served to hold the rail share, it has not had a very large effect on highway cargo movements.

The procedure for estimating annual rail *piggyback* ton-miles in table 1 is represented by the equation:

$$TM_p = O_p \times P \times H \times L,$$

Where,

TM_p = estimated ton-miles of cargo hauled by rail *piggyback*.

O_p = number of *piggyback* rail cars originating (see "Effect of Piggyback Operation on Volume of Highway Truck Traffic," Alan C. Flott, Highway Research Board Record Number 153, 1967).

P = estimated average number of loaded *piggyback* trailers and containers per originating rail car ranging from $P=1.10$ in 1955–59 to $P=1.64$ in 1964.

H = average haul, in miles, estimated for rail *piggyback* cars, based on the ratio of *piggyback* car average in I.C.C. Statement No. 66–1 to the all-flatcar average computed by mileage block from Statement TC–1, then applied to similar flatcar data for 1955–65.

L = average load per loaded *piggyback* semitrailer body or container estimated to be approximately 16 tons based on I.C.C. Statement No. 66–1 and confirmed by 1963 truck weight data for long-haul van body, fully loaded semitrailer combinations.

Although the values and assumptions are based on extensive discussions with experts in the field, the author takes full responsibility for the estimates, which were intended to be realistic but assure a reasonably high *piggyback* ton-mile series.

Table 1.—Trend in the amount and share of intercity cargo transportation by railroad, piggyback, and highway

	1945	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965
Ton-miles, millions:												
Railroad excluding piggyback	684, 148	625, 588	649, 486	619, 721	552, 010	574, 349	568, 638	557, 534	583, 423	610, 486	644, 793	685, 584
Piggyback, TOFC [1] and COFC [2]		1, 305	1, 702	2, 186	2, 524	4, 288	6, 722	8, 761	12, 351	14, 684	17, 296	20, 120
Total railroad	684, 148	626, 893	651, 188	621, 907	554, 534	578, 637	575, 360	566, 295	595, 774	625, 170	662, 089	705, 704
Highway	66, 948	223, 254	248, 846	254, 174	255, 544	278, 934	285, 483	296, 485	309, 407	336, 170	356, 298	388, 438
Total ton-miles	751, 096	850, 147	900, 034	876, 081	810, 078	857, 571	860, 843	862, 780	905, 181	961, 340	1, 018, 387	1, 094, 142
Percent of total ton-miles:												
Railroad excluding piggyback	91. 09	73. 58	72. 16	70. 74	68. 14	66. 97	66. 06	64. 62	64. 45	63. 50	63. 32	62. 66
Piggyback, TOFC [1] and COFC [2]		0. 15	0. 19	0. 25	0. 31	0. 50	0. 78	1. 02	1. 36	1. 53	1. 70	1. 84
Total railroad	91. 09	73. 73	72. 35	70. 99	68. 45	67. 47	66. 84	65. 64	65. 81	65. 03	65. 02	64. 50
Highway	8. 91	26. 27	27. 65	29. 01	31. 55	32. 53	33. 16	34. 36	34. 19	34. 97	34. 98	35. 50
Total percentage	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00	100. 00
Change from previous year, percent:												
Railroad excluding piggyback		13. 29	3. 82	−4. 58	−10. 93	4. 05	−0. 99	−1. 95	4. 64	4. 64	5. 62	6. 33
Piggyback, TOFC [1] and COFC [2]			30. 42	28. 44	15. 46	69. 89	56. 76	30. 33	40. 98	18. 89	17. 79	16. 33
Total railroad		13. 53	3. 88	−4. 50	−10. 83	4. 35	−0. 57	−1. 58	5. 21	4. 93	5. 91	6. 59
Highway		4. 70	11. 46	2. 14	0. 54	9. 15	2. 35	3. 85	4. 36	8. 65	5. 99	9. 02
Total change		10. 90	5. 87	−2. 66	7. 53	5. 86	0. 38	0. 22	4. 91	6. 20	5. 93	7. 44
Piggyback-highway ton-mile percentages:												
Piggyback as percent of other rail		0. 21	0. 26	0. 35	0. 46	0. 75	1. 18	1. 57	2. 12	2. 41	2. 68	2. 93
Piggyback as percent of highway		0. 58	0. 68	0. 86	0. 99	1. 54	2. 35	2. 95	3. 99	4. 37	4. 85	5. 18
Highway as percent of all rail	9. 78	35. 61	38. 21	40. 87	46. 08	48. 21	49. 62	52. 36	51. 93	53. 77	53. 81	55. 04
Indices, 1961—100:												
Railroad excluding piggyback	123	112	116	111	99	103	102	100	105	109	116	123
Piggyback, TOFC [1] and COFC [2]		15	19	25	29	49	77	100	141	168	197	230
Total railroad	121	111	115	110	98	102	102	100	105	110	117	124
Highway	23	75	84	86	86	94	96	100	104	113	120	131
Total	87	99	104	102	94	99	100	100	105	111	118	127

[1] TOFC = trailer on flatcar.
[2] COFC = container on flatcar.

Table 2.—Number of trailer-on-flatcar (TOFC) movements according to four prescribed plans, 1963 [1]

	Plan I [2]		Plan II [3]		Plan III [4]		Plan IV [5]		Totals		
	Origin	Termination	Origin	Termination	Origin	Termination	Origin	Termination	Origin	Termination	Both
State:											
Massachusetts	109	94			175	112			284	206	490
New Jersey	233	245	89	198	248	330	60	68	630	841	1,471
New York	119	123	236	260	155	102			510	485	995
Pennsylvania	116	80	166	176	142	121			424	377	801
Florida					132	357			132	357	489
Illinois	783	723	802	574	615	554	173	215	2,373	2,066	4,439
Ohio			236	135	125	25			361	160	521
Minnesota	36	104	178	194					214	298	512
Missouri			195	257					195	257	452
Texas			129	224					129	224	353
Colorado	142	148							142	148	290
California			371	279			168	152	539	431	970
Percent of waybills	75		65		72		78		68		

[1] Source—appendix C. *Piggyback Traffic Characteristics*, Statement No. 66–1, Interstate Commerce Commission, December 1966; includes only data for States having at least 5 percent of originating or terminating waybills.

[2] Plan I—Railroad transports motor carrier trailers over a portion of the motor carrier's trip. Motor carrier deals with shipper and furnishes pickup and delivery service charging motor carrier rates to shipper.

[3] Plan II—Railroad furnishes all equipment, both flat cars and trailers, and provides pickup and delivery service charging railroad rates to shipper.

[4] Plan III—Railroad transports trailers either owned or leased by the shipper at a flat rate per mile. Shipper is responsible for pickup and delivery.

[5] Plan IV—Railroad transports trailers owned or leased by the shipper on flatcars also owned or leased by shipper at a flat charge per car, loaded or empty. Shipper picks up and delivers, loads and unloads.

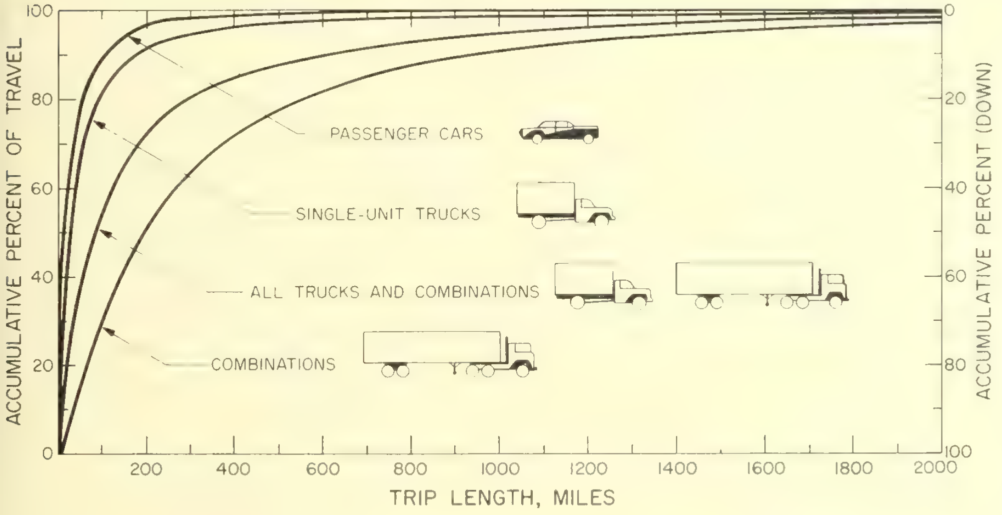

Figure 2.—Accumulative percentage of travel from trips of increasing length for passenger cars, single-unit trucks, and combinations.

Characteristics of Piggyback Operation

The Interstate Commerce Commission report, *Piggyback Traffic Characteristics*,[3] contains extensive information on *piggyback* operations. Data from appendix C of that report was used to compile table 2 in which it is shown that shipments were concentrated in 12 States. Although data relating *piggyback* origins to destinations are not readily available, a substantial proportion of these movements are east-west between the major rail and sea transportation hubs. Because the data in the table are from the Interstate Commerce Commission's (I.C.C.) *1 percent waybill sample*, the total number of shipments can be approximated by multiplying the sample data by 100. Accordingly, it is estimated that 443,900 trailers were shipped by rail to or from Illinois, the State with the maximum number of shipments in 1963. This is an average of 1,216 trailers per day or 51 trailers per hour. Even if all these shipments going through all the *piggyback* terminals in the State had been concentrated on a single major highway, there would have been fewer trailers on that highway than are usually carried by a typical heavy-truck route. On U.S. 66 southwest of Chicago, the number of trailer combinations in 1963 averaged 1,957 vehicles per day. This figure is based on counts obtained by the Illinois Division of Highways.[4] The trailer volume on this one highway was 60 percent more than all TOFC movements originating or terminating in the State.

Several comparisons can be made to determine the portion of present highway truck cargo that might be served by *piggyback*. There would be no advantage to transporting goods by *piggyback* if transportation by tractor truck semitrailer combination alone is less costly than having the trailer moved part of the way by rail. Depending on the orientation of *piggyback* terminals with respect to actual origin and destination of shipments, there are minimum distances for which all-highway hauling is more economical than *piggyback*. "Average short-line length of haul for all *piggyback* rail waybills, was 711 miles per car; the local average haul was 589 miles, and for interline service, 929 miles. Plans I and II had the lowest average short-line hauls per car; plan IV had the greatest. Average short-line hauls ranging 600 miles and upward suggest that *piggyback* competes effectively for relatively long haul traffic. The average hauls computed from the sample exceed distances of 300 to 425 miles which, based on early cost analyses by different transportation interests, were thought to be the shortest distances at which *piggyback* might be competitive with highway operations. They also exceed 1963 average hauls per ton of 464 miles for class I railroads, and the average haul for class I motor common carriers and common carriers of general freight, 267 and 342 miles, respectively." [3]

As shown in figure 2, approximately 37 percent of all travel by all highway trailer combinations consists of trips 300 miles or longer. Only about 27 percent consists of trips longer than 425 miles. As indicated above, the minimum shipping distances for which *piggyback* is likely to be competitive with highway transportation has been estimated to be 300 to 425 miles. More detailed trip-length data by vehicle type is given in table 3. Interpolating between the 300- and 500-mile entries on the line *all trailer combinations*, it is estimated that less than 3 percent of all trips are longer than 425 miles, although these trips account for 28 percent of all travel by combinations. As these are

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[3] *Piggyback Traffic Characteristics*, Statement No. 66-1, Interstate Commerce Commission, December 1966.

[4] *Traffic Characteristics on Illinois Highways, 1963*, Department of Public Works and Buildings, station No. 24BX, p. 100.

Table 3.—Cumulative percentages of trips and travel by different types of vehicles

Vehicle type	Category	One way trip length, miles														
		5	10	20	30	40	50	100	250	500	700	1,000	1,500	2,000	2,500	2,500 and over
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Passenger cars[1]	Trips	56.4	76.8	91.3	95.8	97.3	98.0	99.5	99.9		......	100.0				100.00
	Travel	14.2	31.0	54.5	67.4	73.7	77.8	89.9	97.7		......	100.0			......	100.00
Single unit trucks:[2]																
Panel and pickups	Trips	51.26	75.24	89.29	94.01	96.38	97.61	99.44	99.90	99.97	99.98	99.99	100.00	100.00	100.00	100.00
	Travel	14.38	30.89	50.70	62.22	70.40	75.90	88.08	94.76	96.88	97.59	98.32	98.96	99.36	99.62	100.00
Other 4-tire trucks	Trips	50.14	74.51	88.44	93.46	96.08	97.48	99.44	99.93	99.98	99.99	99.99	100.00	100.00	100.00	100.00
	Travel	13.51	30.00	49.28	61.26	70.17	76.32	89.35	96.34	97.91	98.49	98.78	99.25	99.47	99.67	100.00
2-axle, 6-tire trucks	Trips	40.57	64.09	81.42	88.17	91.99	94.24	98.34	99.77	99.95	99.98	99.99	100.00	100.00	100.00	100.00
	Travel	7.75	18.58	34.69	45.54	54.29	60.93	79.36	93.19	97.04	98.20	98.97	99.55	99.77	99.91	100.00
3-axle or more	Trips	31.40	17.04	79.53	87.26	91.47	93.77	98.25	99.69	99.94	99.97	99.99	99.99	100.00	100.00	100.00
	Travel	5.69	58.85	34.76	46.05	54.74	60.92	79.14	91.80	96.67	97.94	98.88	99.46	99.65	99.84	......
Truck with light trailer	Trips	39.51	59.96	79.81	87.39	90.67	92.06	95.53	98.21	99.17	99.50	99.74	99.89	99.99	100.00	....
	Travel	4.06	8.64	18.85	25.34	29.52	31.83	40.63	56.40	68.90	76.32	83.73	90.68	97.64	99.96	100.00
Subtotal	Trips	45.11	69.17	85.11	90.95	94.10	95.86	98.88	99.84	99.96	99.98	99.99	100.00	....	....	
	Travel	10.08	23.15	40.81	51.98	60.54	66.73	82.74	93.70	96.95	97.95	98.71	99.31	99.60	99.79	100.00
Semitrailer combinations:[2]																
3-axle	Trips	27.39	44.73	59.31	67.53	73.88	78.22	89.20	96.93	99.02	99.46	99.73	99.86	99.92	99.95	100.00
	Travel	1.66	4.23	8.70	13.03	17.73	21.89	38.54	64.23	79.77	85.35	90.24	93.93	95.86	97.63	100.00
4-axle	Trips	18.68	33.02	47.33	54.66	61.24	65.87	81.19	94.30	98.33	99.17	99.71	99.93	99.97	99.98	100.00
	Travel	.77	2.22	5.24	7.83	11.15	14.15	30.15	59.82	79.95	87.25	93.73	96.67	98.61	99.17	100.00
5-axle or more	Trips	18.76	30.84	40.66	47.13	53.73	59.20	76.18	90.67	96.91	98.23	99.05	99.63	99.82	99.94	100.00
	Travel	.56	1.42	2.93	4.59	6.99	9.57	22.08	46.25	69.17	77.27	84.50	92.05	95.63	98.32	100.00
Subtotal	Trips	20.88	35.41	48.66	56.00	62.52	67.29	81.94	94.05	98.15	99.01	99.55	99.84	99.92	99.97	100.00
	Travel	.85	2.28	5.03	7.59	10.82	13.88	28.79	55.91	76.22	83.51	89.98	95.12	97.13	98.62	100.00
Truck and full trailer combinations:[2]																
4-axle or less	Trips	39.88	63.59	78.43	85.05	88.94	91.32	95.88	98.73	99.53	99.75	99.84	99.94	99.98	99.99	100.00
	Travel	4.45	11.56	20.36	26.86	32.16	36.82	50.15	68.82	81.00	86.42	89.44	94.59	97.52	98.76	100.00
5-axle	Trips	12.06	22.02	33.15	41.80	51.83	54.29	78.37	95.57	98.83	99.34	99.84	99.97	99.99	100.00	....
	Travel	.54	1.51	3.83	6.97	11.90	15.43	36.57	73.26	88.96	93.21	97.02	99.16	99.61	99.88	100.00
6-axle or more	Trips	...		10.25	16.84	32.84	34.74	71.25	95.56	99.67	.....	.....	.....			
	Travel			1.85	3.67	10.16	11.08	40.71	81.50	97.22		.....	.....			
Subtotal	Trips	25.24	42.25	55.20	62.81	69.90	17.79	86.92	97.13	99.20	99.55	99.84	99.95	99.98	99.99	100.00
	Travel	1.50	3.96	7.88	11.82	16.94	20.62	40.03	72.35	87.18	91.63	95.21	98.05	99.10	99.61	100.00
Two-trailer combinations:[2]																
5-axle or less	Trips	2.73	7.61	25.53	39.04	47.00	54.00	75.93	95.16	99.04	99.49	99.82	99.96	99.99	100.00	....
	Travel	.12	.62	3.96	8.23	11.78	15.79	35.90	72.06	89.84	93.31	96.83	98.99	99.69	99.95	100.00
6-axle	Trips	23.08	31.36	43.17	62.09	65.69	68.00	85.89	96.84	99.28	99.76	99.92	100.00	.....	.....	..
	Travel	..	1.30	4.35	12.73	15.36	17.56	41.37	73.56	90.31	95.60	98.24	100.00		...	..
7-axle or more	Trips	59.70	64.62	66.52	74.29	78.04	79.40	94.86	99.81	99.99	.....	....	.....		100.00	...
	Travel	1.32	2.60	3.91	11.79	17.03	19.65	65.77	96.07	98.69		.....	.....	..	100.00	..
Subtotal	Trips	7.50	12.65	29.20	42.81	50.20	56.51	77.77	95.55	99.11	99.54	99.84	99.96	99.99	100.00	..
	Travel	.13	.69	3.98	8.54	12.07	15.96	36.74	72.58	90.03	93.53	96.94	99.04	99.69	99.95	100.00
All trailer combinations	Trips	20.66	34.99	48.34	55.93	62.53	67.30	82.10	94.30	98.25	99.06	99.58	99.85	99.93	99.97	100.00
	Travel	.85	2.29	5.11	7.81	11.14	14.26	29.60	57.29	77.26	84.27	90.49	95.40	97.32	98.72	100.00
All trucks and trailer combinations.	Trips	41.58	64.23	79.80	85.90	89.54	91.74	96.46	99.04	99.71	99.85	99.93	99.98	99.99	99.99	100.00
	Travel	5.66	13.18	23.75	30.87	36.93	41.65	57.34	76.30	87.54	91.41	94.78	97.44	98.51	99.28	100.00

[1] Nationwide Automobile Use Study conducted April 1961 for Bureau of Public Roads by Bureau of the Census based on one current population survey panel of approximately 4,000 dwelling units.

[2] 1963 Special Truck Weight Study data obtained by the State highway departments in cooperation with the Bureau of Public Roads in the summer of 1963.

minimum distances for *piggyback* usage, and because the percentages are small for longer distances, it appears that less than 20 to 25 percent of all travel by highway trailer combinations can be considered for eventual diversion to *piggyback*.

Piggyback terminal areas

In or near larger cities, where the majority of shipments originate—either *piggyback* or all-highway hauling—and where the concentrations of the truck combinations are largest, the diversion from highway transportation to *piggyback* theoretically would not reduce the amount of travel by trailer combinations on the highways at all; it would more likely increase the travel by truck combinations on urban streets and highways. The reason for this is that the shipments must be transported by highway to and from the *piggyback* terminal. Why this is likely to occur is shown in figure 3. Shipment A represents a situation in which both origin and destination are within the urbanized area boundary; shipment B represents a situation in which both origin and destination are outside the periphery of the urban area. In situation A, *piggyback* shipment would eliminate the highway travel as shown by the symbol (−), and add the highway travel, as shown by the symbol (+). Careful location analysis would tend to minimize (+) and maximize (−) on the basis of existing and future locations of shipment origins and destinations.

In most areas, choices for *piggyback* terminal locations are limited, as sites where existing rail lines are near freeway interchanges are preferred. Location of plants and warehouses near the *piggyback* terminal can contribute to efficient operation, but commodities must move through, to, and from other parts of the urban area and the surrounding region. Consequently, the moving of commodities by highway to and from ultimate origins and destinations, whether in the *piggyback* container or in another vehicle, cannot be eliminated. Within the urbanized area, the sum of highway travel eliminated, shown in figure 3 by symbol (−), cannot be substantially more than the sum of the added travel to and from the *piggyback* terminal indicated by symbol (+). However, a substantial amount of highway travel between cities can be eliminated.

Shipment B represents a situation in which all-highway movement would not travel through either the origin or destination urban areas. In this situation, the shift to *piggyback* would produce additional urban travel by trailer combination and reduce rural travel. By shifting to *piggyback*, the reduction of the long highway trailer trips would be most noticeable in the more remote rural areas between major terminal cities separated by 300 miles or more. At such locations, these large highway vehicles may constitute 25 percent or more of all traffic; but the total traffic volume is usually quite low.

Spatial and temporal concentrations of urban trailer combination movements may cause problems. In figure 3, the sections identified as *connection* are intended to represent the sections of highways, streets, interchanges, and intersections that provide the connecting links between the *piggyback* terminal and the expressway network. With a limited number of *piggyback* terminals, it will be necessary for *piggyback* shipments from all parts of the urbanized area to traverse these connecting links. The all-highway shipments will traverse a variety of routes throughout the urban area, but in rural areas, the majority will use the principal intercity routes. These spatial dispersions and concentrations have little significance, except that pavements on connecting links must be adequate for the frequent, heavy loads. It is the concentration of trailer combination movements during the periods immediately before departure and after arrival of the rail trains that can create highway capacity problems, particularly if the periods coincide with morning or evening peak hour periods. Because time costs money in commodity transportation, shipppers and consignees often arrange to deliver and pick up their trailers or containers close to the

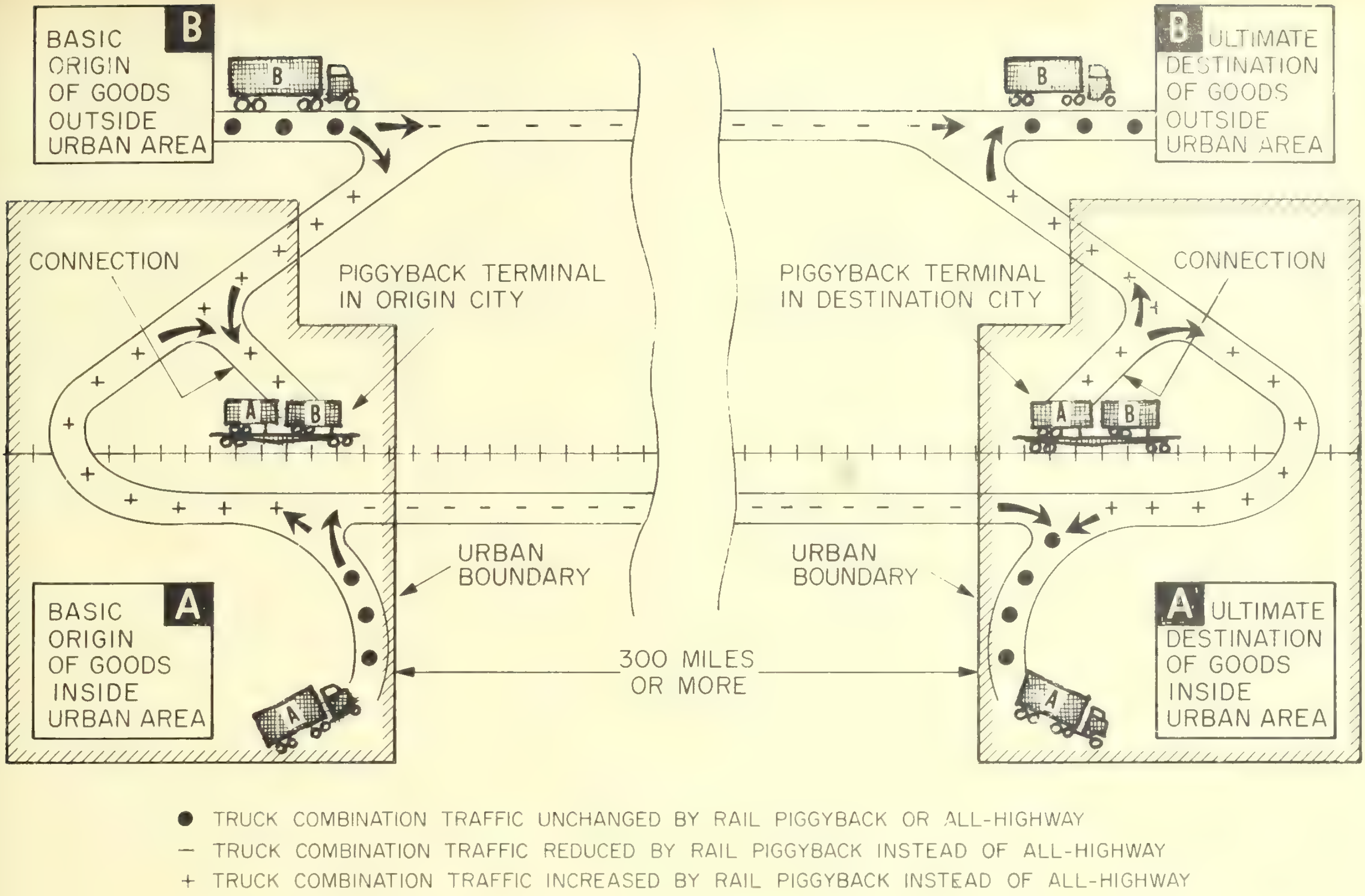
BASIC ORIGIN OF GOODS OUTSIDE URBAN AREA
B
ULTIMATE DESTINATION OF GOODS OUTSIDE URBAN AREA
B
CONNECTION
PIGGYBACK TERMINAL IN ORIGIN CITY
PIGGYBACK TERMINAL IN DESTINATION CITY
CONNECTION
A
B
A
B
URBAN BOUNDARY
URBAN BOUNDARY
BASIC ORIGIN OF GOODS INSIDE URBAN AREA
A
ULTIMATE DESTINATION OF GOODS INSIDE URBAN AREA
A
300 MILES OR MORE
A
A
● TRUCK COMBINATION TRAFFIC UNCHANGED BY RAIL PIGGYBACK OR ALL-HIGHWAY
— TRUCK COMBINATION TRAFFIC REDUCED BY RAIL PIGGYBACK INSTEAD OF ALL-HIGHWAY
+ TRUCK COMBINATION TRAFFIC INCREASED BY RAIL PIGGYBACK INSTEAD OF ALL-HIGHWAY

Figure 3.—Hypothetical changes in highway travel patterns caused by **piggyback.**

scheduled train time. As unloading rates exceed one trailer per minute, it is not unlikely that several semitrailer combinations will be added to the traffic stream during each signal cycle or equivalent time interval at certain connecting link intersections. If near-capacity traffic already exists at the intersections, serious congestion may result. Because excessive delays would tend to discourage use of the *piggyback* terminal, it is important that adequate traffic capacity be provided on the connecting links.

Accordingly, rail *piggybacking*, although tending to relieve the rural portions of intercity routes by as much as 25 percent of travel by trailer combinations, will have little effect on the amount of such travel in the urban areas served by the terminals. Some increase in total urban travel and the possible concentration of trailer combination movements near *piggyback* rail terminals warrants the attention of transportation planners. Additional lanes of highway for increased capacity at the terminals is one problem. If from one to no more than a half dozen combinations each minute—as limited by rail loading and unloading—entered or left a highway that serves a *piggyback* terminal, a level 4-lane expressway would experience added congestion during peak hours. A more critical problem is the effect on an adequate 4-lane urban expressway of arrivals and departures during off-peak hours. The need is for a sufficiently high level of service on all sections of the urban arterial network so that cargo can be moved expeditiously between the *piggyback* terminal and the ultimate origins and destinations.

Legal and regulatory considerations

A decision by the Interstate Commerce Commission in 1954 signaled the succeeding rapid growth of *piggybacking*.[5] In 1967, a Supreme Court decision established the right of motor carriers to avail themselves of railroad *piggyback* services at the same rates charged to other customers,[6] which should encourage *piggyback* to be used on a larger scale for shipments of sufficient length. As indicated previously, this increased use could affect a maximum of about 25 percent of all combination vehicle miles. To approach this maximum would require railroads to be properly located to serve all these long truck movements.

The point at which *piggyback* becomes economical is related to the distance that a trailer unit can be moved on highways during one 8-hour work shift. While practices vary according to the type of operation and geographic location, it is evident that once a trailer or semitrailer is connected to a power unit and is underway, the cost for each additional mile driven is small. The owner's costs for capital investment in the vehicle and the driver wages are the same whether the vehicle is being hitched, driven to a *piggyback* terminal, unhitched, returned empty, or continuing down the road toward the destination. Usually, the driver must be employed for a full shift even if only one or two units are to be moved. When several units are to be loaded at about the same time, near the end of a work shift and shortly before scheduled departure of the trailer train, then several power units may be required to move them all to the loading point on time. If these power units must stand idle except for the few hours required each day for moving trailers to and from the loading point, it may be more economical to run some or all of the combinations through to the destination.

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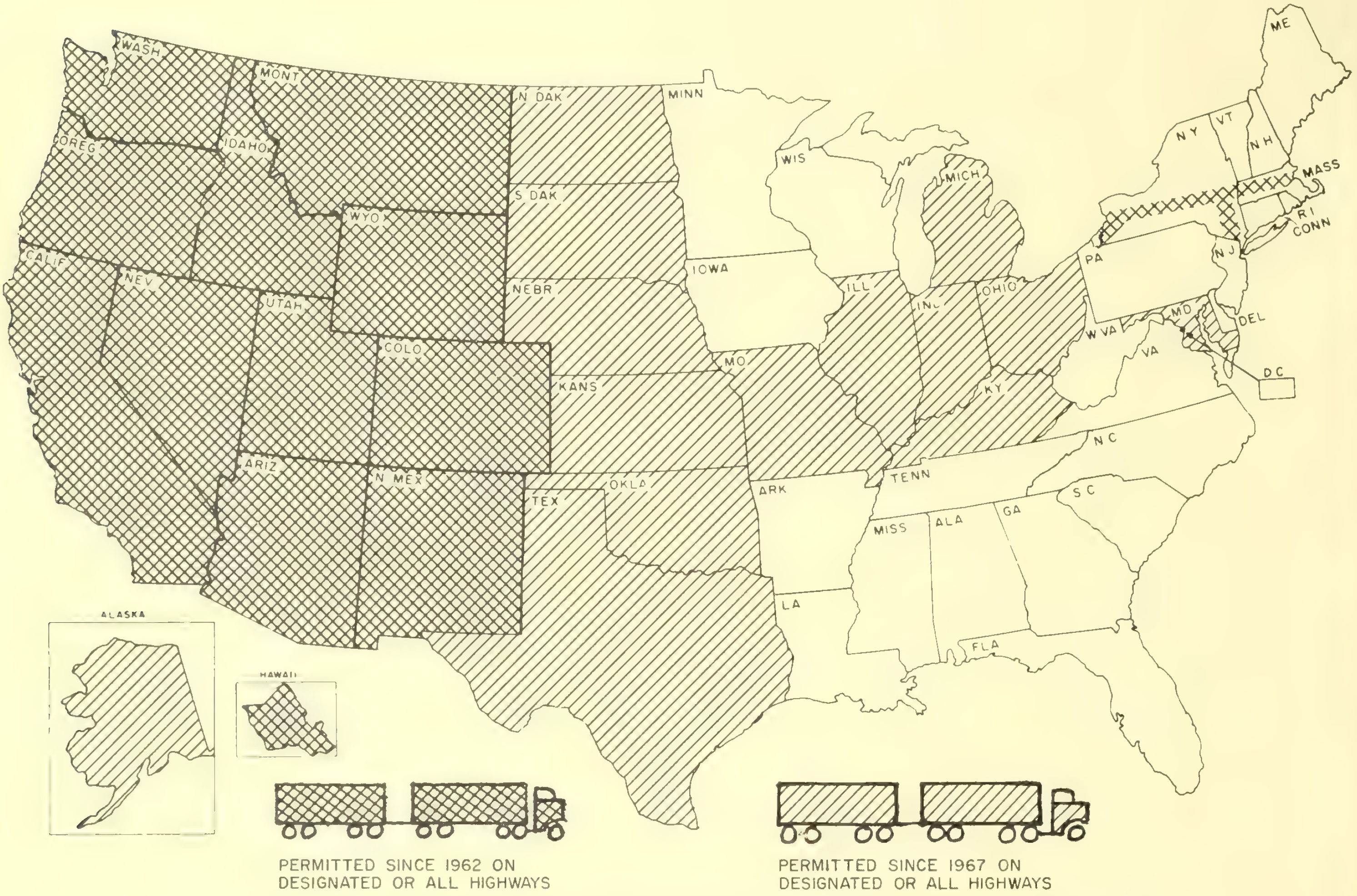
[5] *The New Haven Case*, Interstate Commerce Commission Docket No. 31375, Movement of Highway Trailers by Rail, 293, Interstate Commerce Commission 93, July 30, 1954.

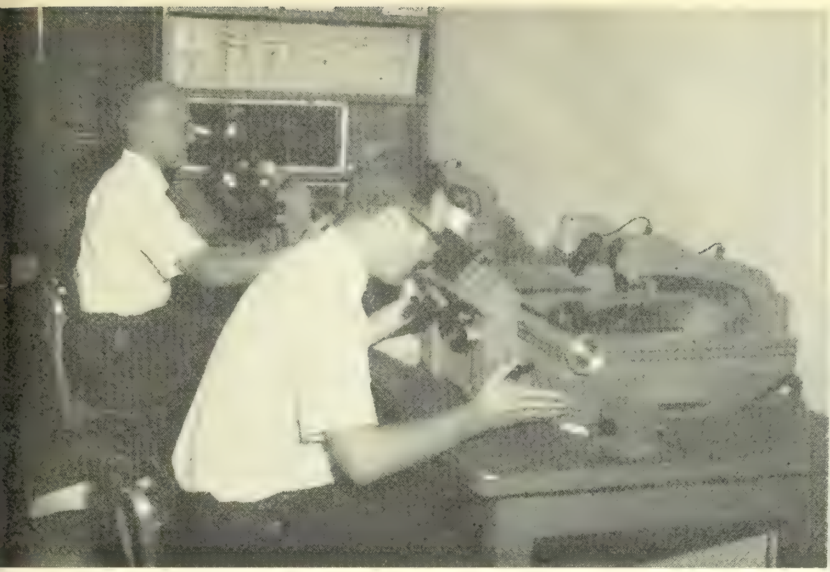
[6] No. 57, American Trucking Association, Inc., et al., versus the Atchison, Topeka and Sante Fe Railway Company, et al.; No. 59, National Automobile Transportation Association of Detroit, Mich., versus Atchison, Topeka and Santa Fe Railway, et al.; and No. 60, United States, et al., versus the Atchison, Topeka and Santa Fe Railway Company, et al.

The capital cost of the power unit and the driver wages represent a large part of the cost of highway cargo movement, and substantial economies can be achieved when more cargo can be moved by a single power unit. Recent changes in State laws have brought about the changes shown in figure 4. The States shown with dark shading have permitted the use of combinations with a semitrailer and full trailer 65 feet in length since January 1964. The lightly shaded States have subsequently revised their laws and now permit use of these vehicles. In many States these large combinations are restricted to travel only on the major highways, including the Interstate System. In New York and Massachusetts, the combinations are permitted only on certain toll roads, as indicated by the shading. Hence, 4 years ago these large combinations, called *double bottoms*, were permitted in only a few areas; today, except for the Appalachian barrier, it is legal to drive them almost coast to coast. The increased use of *double bottoms* has increased, by 30 to 50 percent, the amount of cargo that can be transported by a single power unit and driver. Additional economies result for operations in which a full load in a smaller body increases loading and other efficiencies. These changes increase the economy of long distance highway transportation.

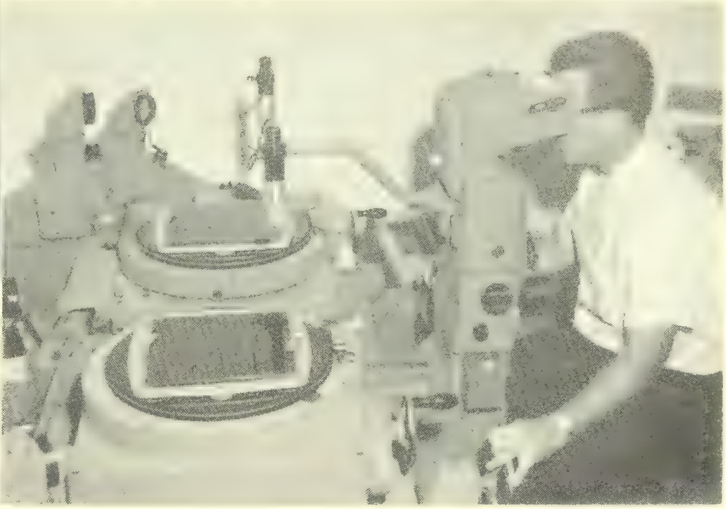


IN IND., KANS., MASS., N.Y., AND OHIO, FULL COMBINATION HAS BEEN PERMITTED ON TOLL ROADS SINCE 1962.

Figure 4.—States that have permitted use of tractor semitrailer-full trailer combinations since 1962 and currently (from data summarized by American Trucking Association).



Two of the photogrammetric instruments, point transfer device (left) and sterocomparator (below), that were used in the evaluation presented in this article. The point transfer device was used to drill pass points on photographic plates; the sterocomparator was used to measure photogrammetric x and y plate coordinates.



Analytic Aerial Triangulation for Highways— A Comparison of Two Methods

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by [1, 2, 3] JESSE R. CHAVES,
Highway Engineer
Engineering Systems Division

Results of a comparison of two methods of analytic aerial triangulation are presented in this article. Accuracies of the Canadian National Research Council (NRC) and the U.S. Coast and Geodetic Survey (CGS) systems are compared. The evaluation was performed with 1:4,800- and 1:9,600-scale photographs. Strip coordinate computations and strip adjustments for the two methods were tested using the same measured plate coordinates and ground control.

Although the error propagation within each strip computation is undoubtedly different, the resultant computed ground coordinates are not significantly affected. Either the NRC or the CGS method of analytic aerial triangulation will provide acceptable results for highway location and design.

Introduction

ANALYTIC aerial triangulation has been established as a useful method of obtaining accurate topographic data that can be used in highway planning, location, and design, and several methods have been developed for this approach to photogrammetric bridging. Two of the more prominent methods for computing the positions and elevations of points on the ground, those developed by the National Research Council of Canada (NRC) (*1, 2*)[4] and the U.S. Coast and Geodetic Survey (CGS) (*3, 4*), were evaluated in the investigation reported here. The coordinates obtained from either method can be used for many purposes in highway location and design, but State highway personnel and others who are concerned with photogrammetry and are interested in this method of bridging often question the accuracy of the methods.

Two scales of photographs were used in the investigation to evaluate the accuracy of computed ground coordinates, and the errors in the two methods were compared and evaluated. The information presented should assist State highway organizations and others in selecting the method of analytic strip triangulation that will best suit their individual requirements.

Two previous investigations[5] (*5*) have been conducted to determine the feasibility of

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[1] Mr. Chaves was with the Office of Engineering and Operations when the project reported here was initiated.

[2] Presented at the Annual Convention of the American Society of Photogrammetry, Washington, D.C., March 1968.

[3] Previously published by Photogrammetric Engineering, vol. XXXIV, No. 7, July 1968, pp. 697–704.

[4] Italic numbers in parentheses identify the references listed on p. 126.

[5] *Survey Control Extension by Analytic Aerotriangulation for Highways*, by J. R. Chaves, unpublished thesis, Syracuse University, September 1965.

using analytic aerial triangulation in highway engineering. These included evaluating computed ground coordinates for supplemental control for large-scale, small contour interval mapping employed in highway location and design. In the previous work, coordinate measurements were made with monocular comparators. In this investigation, a stereocomparator was used.

Aerial Photography

Aerial photographs, used in a mapping project for the extension of Colonial Parkway near Williamsburg, Va., were provided by Regional Office 15 of the Federal Highway Administration. Two photographic flight strips, used for the comparative evaluation experiment, were taken with a Wild RC–8 mapping camera equipped with a 6-inch focal length Aviogon lens. The first flight strip of an area about 12,000 feet long consisted of 10 photographs at a scale of 1:4,800. The second flight strip of an area about 16,000 feet long, consisted of six photographs at a scale of 1:9,600. The larger scale photographs were used for map compilation and bridging; the smaller scale photographs were used exclusively for analytic bridging. Diapositive plates were printed emulsion-to-emulsion from the aerial negative film by an automatic-dodging printer.

Ground Control Survey and Photographic Targets

Basic horizontal control was surveyed to better than second-order accuracy using a Tellurometer and Wild T–2 theodolite; vertical control was surveyed to second-order accuracy using a Zeiss N–2 automatic level. Ground positions of 39 points and elevations of 42 points were surveyed on the ground. Ground coordinates of these points were available for controlling the triangulated strips and for testing the accuracy of the analytically computed coordinates.

All but four surveyed points were premarked by photographic targets. The other four points were natural objects that could be readily identified in the aerial photographs. Three types of target designs (fig. 1) were used as markers of surveyed ground control. Nine of the type A targets were placed throughout the photographed area by the mapping contractor. Seven targets of the type B and 23 of the type C targets were placed throughout the project by personnel of the Bureau of Public Roads. The legs of the type A targets were made of white muslin. The centers of the type B targets consisted of alternating colored cloth wedges of either blue and black or brown and black. The centers of the type C targets were solid black squares. Whenever targets were placed in wooded areas, the legs were extended somewhat to facilitate locating them on the aerial photographs.

Photogrammetric Instruments and Measuring Procedure

Photographic x and y plate coordinates were measured with a Wild STK 1 Sterocomparator equipped with a Wild EK 4 Electric Coordinate Printer that recorded coordinate measurements to the nearest even micron. Comparator output was recorded on punched cards that could be used as computer input data. Measurements were made at 70° F. ± 1° in a temperature controlled room using 11 diameter magnification and a 40-micron diameter measuring mark.

A Wild Pug 3 Point Transfer Device was used to drill six pass points, perpendicular to the flight axis, along the center of each photographic plate. Two holes were drilled in the vicinity of the customary pass-point locations. Wherever feasible, these were located in areas of relatively flat topography so that x and y parallaxes at the time of coordinate measurement could be accurately and readily removed. Although a three-dimensional view was available for selection of pass-point locations, all holes were actually drilled monocularly, using the same drill.

During measurement, each plate was oriented with its emulsion side down and the photographic x-axis nearly parallel to the comparator x-axis. The x and y coordinates were measured on the left-hand stage while the parallaxes, px, and py, were recorded from the right-hand stage.

Each of the image points was measured four times. Because the fiducial marks in the camera had open centers, it was necessary to measure each of the four legs and then mathematically intersect for the center of the fiducial. Five measurements were made on each leg.

Analytic Systems

It is beyond the scope of this article to present the mathematical basis of the analytic aerial triangulation systems used in the investigation; however, the theoretical basis can be pursued by consulting the referenced literature (*1, 2, 3, 4, 6, 7, 8, 9*). Documented FORTRAN computer programs are also included in the referenced literature (*1, 2, 3, 4, 9*).

Computers

Because of their availability, four IBM computers, were used in the investigation: 7030 (STRETCH), 7010 (60K), 7090 and 360 Model 50. To avoid unnecessary delay, no computer program conversions were attempted during the investigation but all programs are now operational on the IBM 360.

The STRETCH computer used for computing the strip coordinates by the triplet method was made available through the courtesy of the CGS. The NRC strip computations were performed on the IBM 7010 system using an 18-digit mantissa. The CGS strip adjustments were computed on the IBM 7090 and the NRC strip adjustment with the IBM 360.

Computer Program Features

For the CGS strip computation photographic x and y coordinates of each image point occur on separate cards. Measurements from one to 10 can be made for each point, but all the cards must be together. In multiple observations, coordinates that deviate more than 25 microns from the mean are rejected. If two such rejections occur in a given set, the computation is stopped and a new triangulation is started. Coordinates of pass points for each photograph of the triplet must be in a

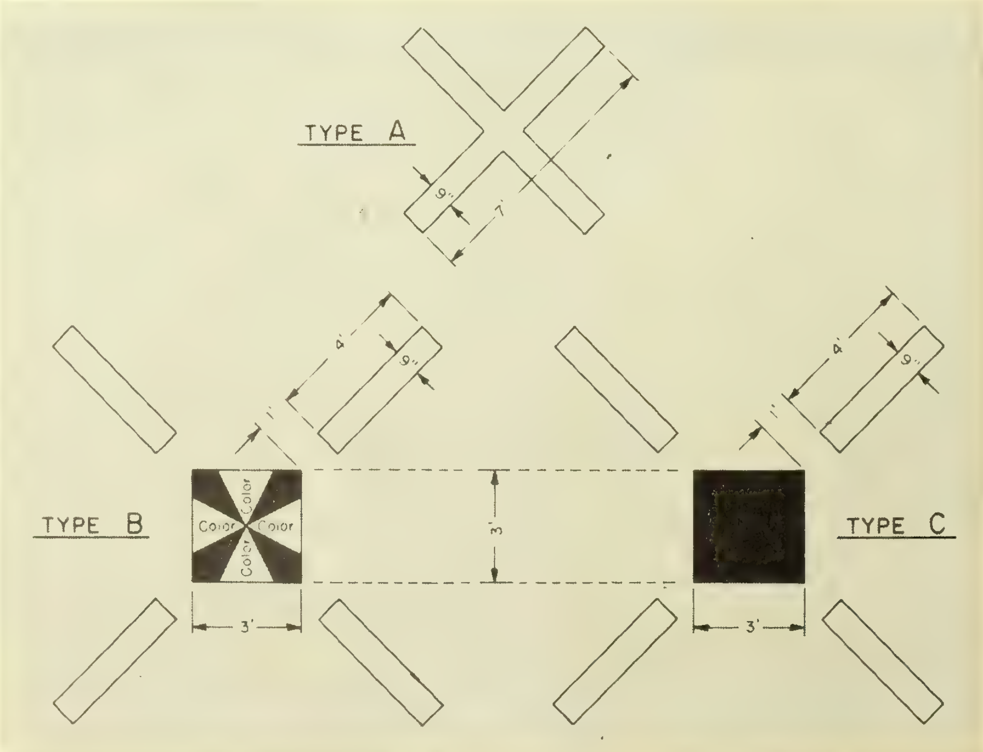

Figure 1.—Photographic target designs.

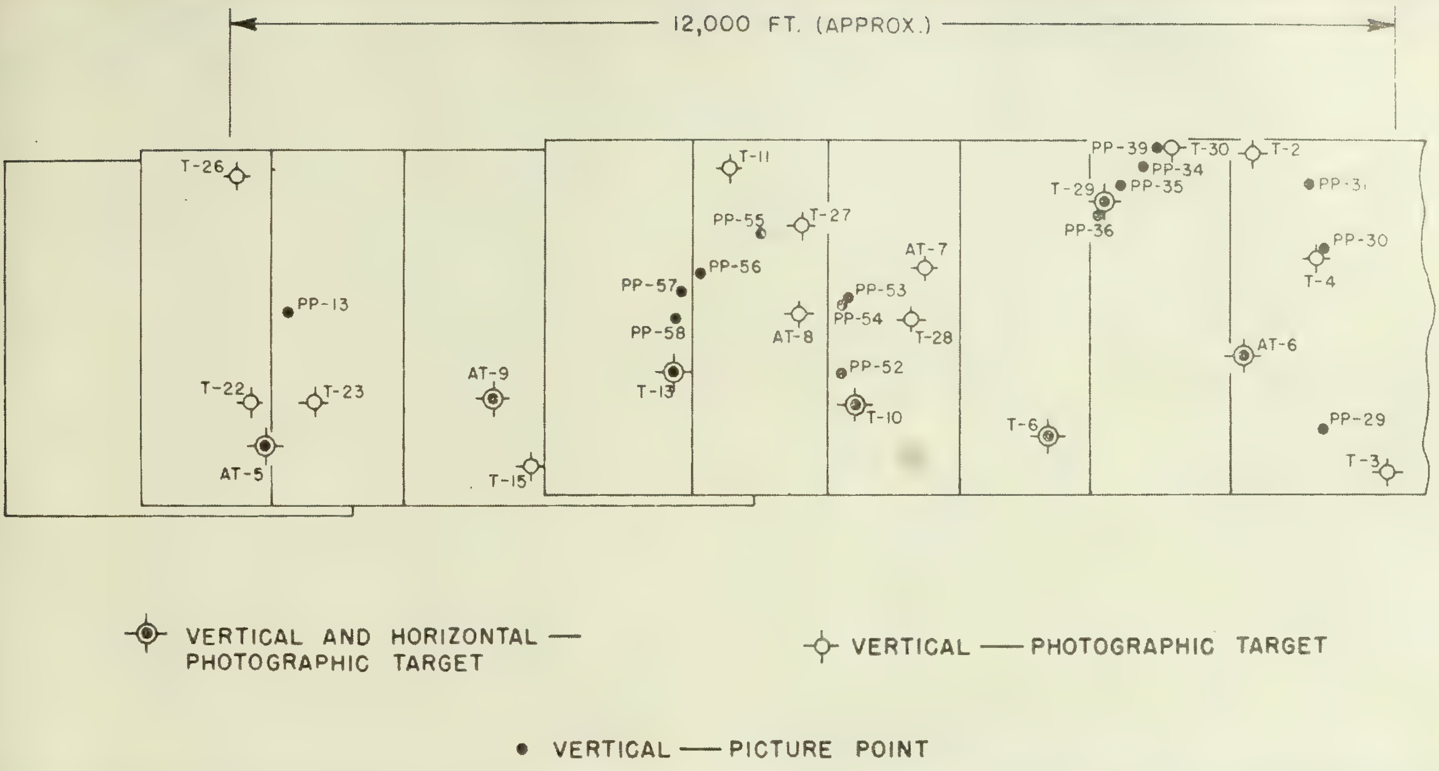

Figure 2.—Ground control distribution, 1:4,800-scale photography.

strict sequence, and a card sort is performed to insure the proper order. In the triplet solution, a given pass-point image is rejected if its residual parallax exceeds the limit set by the user. The companion pass point in the same area is then substituted for it. If two pass points, for a given model in the same area are rejected, the solution is terminated.

The vector sum of all x and y parallaxes is printed out for each pass point that appears on three photographs; only the y parallaxes are output for all other image points. A single root-mean-square value of all residual parallaxes for the 18 pass points in a triplet is also output and serves as a reliability number for the triplet.

The NRC strip computation provides for correcting the measured photographic coordinates for the effects of differential film shrinkage, radial lens distortion, earth curvature, and atmospheric refraction.[6] Two correction factors for differential film shrinkage are applied in the x and y directions. This single set of values is applied to all the photographs in the flight strip. Any number of image points may be used for relative orientation, and an experimental weighing equation may be applied if the photographs have been obtained with a Wild 6-inch Aviogon lens. With this equation, image points near the principal point are given more weight in the relative orientation solution than those located near the corners of the photograph. As many as 10 image points may be used for scaling by an appropriate signal on each scaling point card. Equal weight is given to each scaling point. There are also four standard patterns for the scaling points that can be used, depending on a number punched in the first data card of the strip. This same pattern of image points is used throughout the triangulated strip, but a maximum of four scaling points is permitted with the standard patterns. The program has provisions for discarding anomalous scale transfer points.

The measured photographic coordinates for input to the NRC program are arranged in groups according to models. The first card of each model contains the coordinates of the princiapl points of the two photographs and a number that determines the number of points to be used in relative orientation. The coordinates of corresponding image points appear on each of the subsequent cards. Immediately following the first card are cards for the relative orientation points. All other object-point cards come last. The residual parallaxes will be at photograph scale, providing the value of the base component bx in the first model has been set equal to the actual distance on the photograph. Residual y parallaxes are printed out for each image point.

Input data for the CGS and NRC strip adjustment programs are similar. They include:

- Strip coordinates and surveyed ground control data.
- Strip coordinates of all the points whose ground coordinates are needed.
- The x and y strip coordinates of two points near each end of the flight strip for defining the axis-of-flight.
- A card containing the degrees of polynomials to be used.

The CGS and NRC methods provide for first, second, and third degree polynomials for correcting the horizontal and vertical coordinates. The NRC program also allows for higher degree polynomials and the use of a separate degree polynomial for correcting scale and azimuth, longitudinal tilt, and transversal tilt. The NRC program can also be used for a block adjustment of parallel overlapping flight strips.

Evaluation Scheme

In testing the accuracy of the analytic computations by the two methods, the following procedure was employed:

- The measured x and y plate coordinates for the 1:4,800- and 1:9,600-scale photographic strips were corrected for film and radial lens distortion. The mathematical formulation is described in reference (*8*). This method of coordinate refinement is included as an integral part of the Three-Photo Aerotriangulation program (*3*).
- The strip coordinates for the two flight strips were computed by the NRC and CGS methods using the same set of refined coordinates. Twelve image points in each model

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[6] The coordinate refinement portion of the NRC program was not used in the investigation.

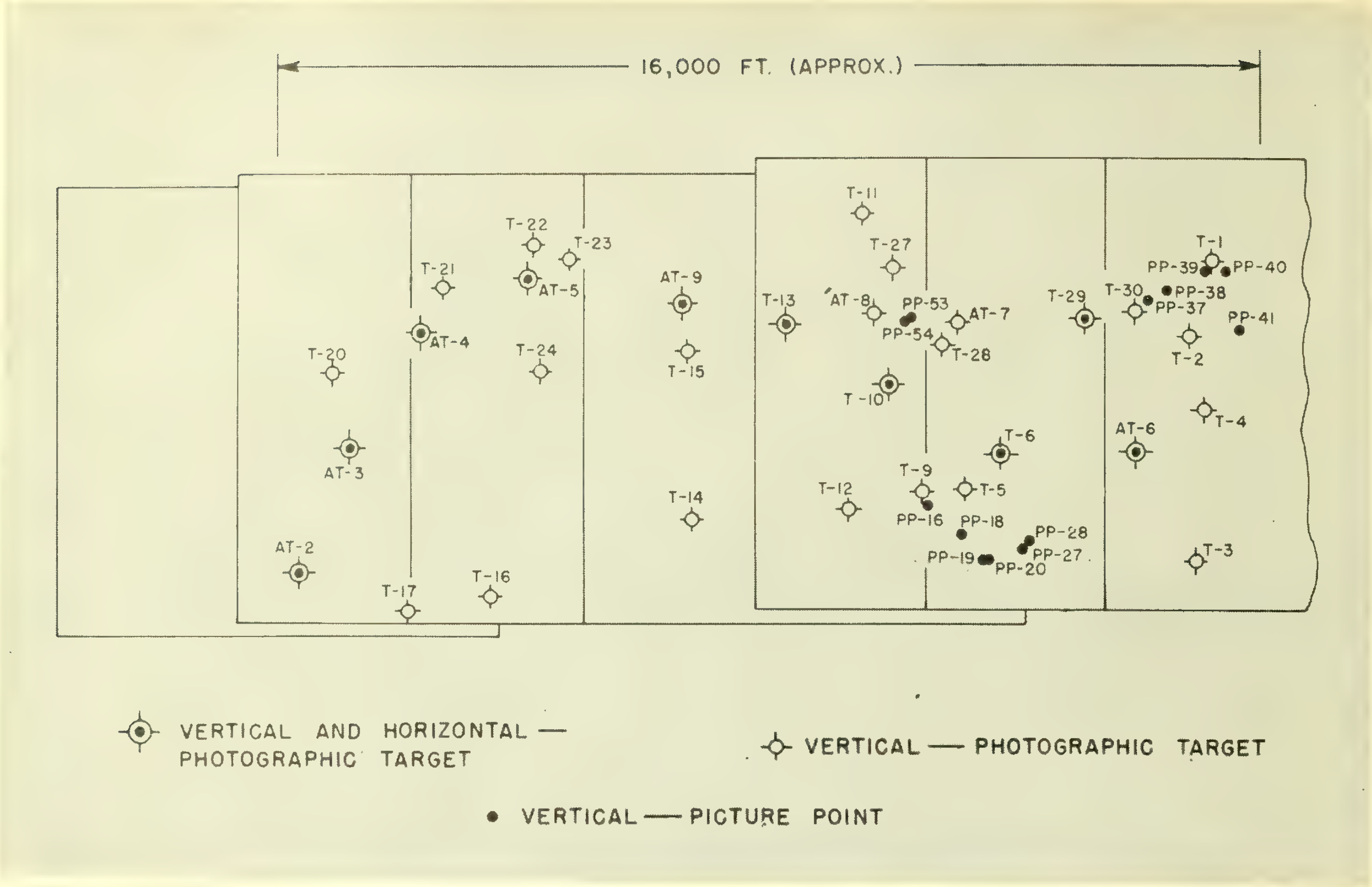

Figure 3.—Ground control distribution, 1:9,600-scale photography.

were used to compute the relative orientation. Two of these image points were located in each of the six conventional pass-point locations.

- The computed strip coordinates for each scale of photographs were adjusted and transformed to ground coordinates by the NRC and CGS strip adjustment methods. Second and third degree polynomial adjustments were employed for each of four combinations. The following four combinations of strip coordinates and strip adjustments permitted a comparison between two independent methods of analytic triangulation and enabled determination of the differences caused by either the strip adjustments or the strip coordinate computation:

NRC-NRC—NRC strip coordinates and NRC strip adjustment.

CGS-CGS—CGS strip coordinates and CGS strip adjustment.

NRC-CGS—NRC strip coordinates and CGS strip adjustment.

CGS-NRC—CGS strip coordinates and NRC strip adjustment.

The standard errors for the *X*, *Y*, and *Z* coordinates were computed for all ground surveyed test points that were withheld from the strip adjustment solution.

The ground control distribution used for controlling and testing the computations for the 1:4,800-scale photographs is shown in figure 2. Four horizontal and seven vertical control points were used for strip adjustment. The horizontal ground control points used are identified in the figure as T–26, AT–9, T–29, and T–4. Vertical control is designated as AT–5, T–26, T–11, T–13, T–28, T–2, and T–3. The ground control distribution for the 1:9,600-scale photographs is shown in figure 3. Four horizontal control points (AT–3, AT–9, AT–7, T–4) and eight vertical control points (AT–4, T–17, T–22, T–9, T–11, T–28, T–1, T–3) were used to adjust this strip.

The *X* and *Y* ground coordinates of 14 points and the elevations of 13 points were available for testing the accuracy of analytically computed coordinates for the 1:4,800-scale flight strip. The horizontal position of 25 ground points and elevations of 23 points were available for testing the computed coordinates for the 1:9,600-scale strip.

Results

Triangulation—1:4,800-scale photographs

The standard errors obtained from the different combinations of strip coordinate and strip adjustment computations for the 1:4,800-scale photographs are shown in tables 1 and 2. By the two independent methods, using second degree polynomials, no significant differences in the computed *X* and *Z* coordinates were evident in comparison No. 1, table 1. The standard error of the *Y* coordinates for the CGS method is slightly smaller than that for the NRC method. Similarly, in the second comparison, it is slightly smaller than that for the NRC method. In the second comparison, only a slight reduction in the standard error of *Y* was caused by the CGS adjustment. In comparison No. 3, no significant differences resulted from the strip adjustments.

In the fourth and fifth comparisons, no differences could be attributed solely to the strip computations. Any minor differences that may have existed from the strip coordinates alone are likely to have been compensated by the adjustment procedure. In all trial combinations the standard errors for *Y* were less than those for *X*.

In the first comparison shown in table 2, the CGS computation, compared with the NRC method, resulted in standard errors that were larger in *X* and smaller in *Y* and *Z*. In comparisons Nos. 2, 3, 4, and 5, the standard errors for *Z* suggest that the slight improvement in the standard error of the *Z* coordinates in comparison No. 1 may have been due to the CGS strip computation rather than to the strip adjustment. A similar

Table 1.—Standard errors for computed ground coordinates from second degree strip adjustments, scale—1:4,800

Comparison number	Trial identification [1]	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
1	NRC-NRC	0.49	0.38	0.34
	CGS-CGS	.51	.27	.32
2	NRC-NRC	.49	.38	.34
	NRC-CGS	.48	.30	.34
3	CGS-CGS	.51	.27	.32
	CGS-NRC	.47	.32	.30
4	NRC-NRC	.49	.38	.34
	CGS-NRC	.47	.32	.30
5	NRC-CGS	.48	.30	.34
	CGS-CGS	.51	.27	.32

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

Table 2.—Standard errors for computed ground coordinates from third degree strip adjustments, scale—1:4,800

Comparison number	Trial identification [1]	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
1	NRC-NRC	0.46	0.52	0.44
	CGS-CGS	.61	.31	.37
2	NRC-NRC	.46	.52	.44
	NRC-CGS	.60	.34	.44
3	CGS-CGS	.61	.31	.37
	CGS-NRC	.48	.47	.39
4	NRC-NRC	.46	.52	.44
	CGS-NRC	.48	.47	.39
5	NRC-CGS	.60	.34	.44
	CGS-CGS	.61	.31	.37

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

Table 3.—Error differences resulting from quadratic and cubic polynomials, scale—1:4,800

Trial identification [1]	Polynomial	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
NRC-NRC	Quadratic	0.49	0.38	0.34
NRC-NRC	Cubic	.46	.52	.44
CGS-CGS	Quadratic	.51	.27	.32
CGS-CGS	Cubic	.61	.31	.37

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

analysis of comparisons Nos. 2, 3, 4, and 5 shows that the more accurate *Y* coordinates of the CGS method were primarily due to the CGS strip adjustment. The smaller standard error for *X* in comparison No. 1 was largely a result of the NRC strip adjustment.

For all the trials adjusted by the NRC method, the standard errors for *X* were about the same as those for *Y*, whereas the *Y* coordinates from the CGS strip adjustments were about twice as accurate as the *X* coordinates.

The differences in standard errors resulting from the quadratic and cubic polynomials used in the two strip adjustments are shown in table 3. For both the NRC and CGS method, the second degree strip adjustments gave better overall results, as expected for a relatively short flight strip and dense ground control.

Triangulation—1:9,600-scale photographs

The standard errors for computed ground coordinates using the 1:9,600-scale flight strip appear in tables 3 and 4. Strip adjustments were performed using second degree polynomials for the comparisons shown in table 3.

The CGS method gave slightly lower standard errors for *Y* and *Z* in comparison No. 1. In comparison No. 2, no significant differences between the computed coordinates resulted from the two strip adjustments. For comparison No. 3, the results do not corroborate those of the previous comparison because the CGS strip adjustment gave slightly smaller standard errors for *X*, *Y*, and *Z*. In the fourth comparison, no significant differences were caused by the method of strip computation. It is suggested in comparison No. 5 however, that the CGS strip computation gave slightly more accurate elevations.

In both the NRC and CGS strip adjustments, standard errors for *X* and *Y* were about equal. This is unlike the standard errors from second degree CGS adjustments for the 1:4,800-scale photographs (table 1) in which the *Y* coordinates were computed more accurately than the *X*.

Standard errors for the ground coordinates, using third degree polynomials, are shown in table 4. In comparison No. 1, the CGS method gave markedly improved *X* and *Y* coordinates and only slightly improved *Z* coordinates. In the second and third comparisons, the improvements in *X* and *Y* were due mainly to the CGS strip adjustment. In these comparisons, it is suggested by the data for the *Z* coordinates that improvement was a result of the CGS strip adjustment. In comparison No. 4 there appeared to be only slight improvement in *X* from the NRC strip computation. In comparison No. 5 a similar indication was shown for *X*, but improvements for *Y* and *Z* were also shown because of the NRC strip coordinates.

It is suggested in comparisons Nos. 4 and 5 (tables 3 and 4) that the third degree NRC strip adjustment has simply *adjusted out* any differences that may have existed between the strip coordinates. The CGS adjustment does this to a lesser extent. It is also possible that the NRC strip computations were more accurate for the 1:9,600-scale strip, but the improvement was insignificant in terms of the coordinate improvements resulting from the CGS strip adjustment (comparison No. 1). Comparisons for the 1:4,800-scale strip (tables 1 and 2) do not, however, substantiate this conclusion. In table 1, no significant differences were indicated between the two methods of strip computation, whereas in table 2, it is suggested that the slight improvement in *Z* coordinates was due to the CGS strip computation.

Both the NRC and CGS strip adjustments resulted in standard errors for *X* that were about equal to those for *Y*. This is the same relationship obtained with the quadratic adjustments. Differences obtained from quadratic and cubic polynomial adjustments for the two independent methods of triangualtion are shown in table 6.

A significant difference occurred in the standard errors of the *X* and *Y* coordinates owing to the degree of polynomial for the NRC adjustment. There was little or no difference for the horizontal coordinates regardless of whether a second or third degree CGS adjustment was employed. For both the NRC and CGS methods, slightly lower standard errors for *Z* were obtained with second degree adjustments.

Table 4.—Standard errors for computed ground coordinates from second degree strip adjustments, scale—1:9,600

Comparison number	Trial identification [1]	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
	NRC-NRC	0.58	0.61	0.62
	CGS-CGS	.54	.51	.48
2	NRC-NRC	.58	.61	.62
	NRC-CGS	.55	.58	.62
3	CGS-CGS	.54	.51	.48
	CGS-NRC	.60	.64	.58
4	NRC-NRC	.58	.61	.62
	CGS-NRC	.60	.64	.58
5	NRC-CGS	.55	.58	.62
	CGS-CGS	.54	.51	.48

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

Table 5.—Standard errors for computed ground coordinates from third degree strip adjustments, scale—1:9,600

Comparison number	Trial identification [1]	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
1	NRC-NRC	0.89	0.92	0.66
	CGS-CGS	.55	.45	.56
2	NRC-NRC	.89	.92	.66
	NRC-CGS	.45	.33	.47
3	CGS-CGS	.55	.45	.56
	CGS-NRC	.97	.92	.66
4	NRC-NRC	.89	.92	.66
	CGS-NRC	.97	.92	.66
5	NRC-CGS	.45	.33	.47
	CGS-CGS	.55	.45	.56

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

Table 6.—Error differences resulting from quadratic and cubic polynomials, scale—1:9,600

Trial identification [1]	Polynomial	Coordinate errors		
		X	*Y*	*Z*
		Feet	*Feet*	*Feet*
NRC-NRC	Quadratic	0.58	0.61	0.62
NRC-NRC	Cubic	.89	.92	.66
CGS-CGS	Quadratic	.54	.51	.48
CGS-CGS	Cubic	.55	.45	.56

[1] NRC—National Research Council; CGS—U.S. Coast and Geodetic Survey.

Photographic Images

The discussion here largely reflects the observations made by the stereocomparator operator during measurement of the two photographic flight strips used in the investigation. The target designs used in the investigation are illustrated in figure 1. Based on the use of a 40-micron diameter measuring mark (black dot) and 11 diameter magnification, the center of the type *C* target was found to be both too large and too dark for precise measurement. This was true for both the 1:4,800- and 1:9,600-scale photographs. There was a tendency for the black measuring mark to disappear from view within the target center. Under these circumstances, the type *C* target did not appear suitable for analytic triangulation.

The colored wedges forming the center of the type *B* target lacked tonal contrast in the aerial photograph. The center of this type of target had a rather uniform, gray photographic tone and appeared more like a type *C* target except for the lighter tone. The type *B* target is preferred to type *C* because the black floating mark can be seen within the target center.

The center of the type *A* target appeared too large for optimum measuring accuracy on both scales of photographs. It has been suggested that reduction of the leg widths would provide a more suitable target for the two scales of photographs tested.

In general, the picture points selected by the field crew were found acceptable for coordinate measuring. It was impossible to establish any definite correlation between the types of images (picture points or target types) and the errors in position and elevation at these points.

Summary and Conclusions

Differences in computed ground coordinates of the two analytic systems are caused primarily by the strip adjustments. The degree of polynomial used produces significantly different results with the NRC strip adjustment, but it has a lesser effect with the CGS adjustment. Second degree polynomials in both methods gave the better overall results. With the NRC strip adjustment, polynomials higher than second degree appear unwarranted. For the CGS strip adjustment, the degree of polynomial used should depend on the scale of photographs, length and density of flight strip, and distribution of ground control.

Although the error propagation within each strip computation is undoubtedly different, the resulting computed ground coordinates are not significantly affected.

The magnitude of the standard errors obtained for the computed ground coordinates in the investigation do not necessarily represent the utmost in accuracy that can be expected for the scales of photographs and the distributions and densities of ground control used. Regarding optimum densities and distributions of ground control, no conclusions of the effect of using different photographic target designs are possible.

The author believes that the CGS method of compensating for film distortion is superior to the NRC method. In the NRC program, the same average linear scale factors are applied to the entire flight strip. The CGS method, however, is applicable only to cameras with four corner fiducials, or eight fiducials, if additional fiducials are present along the mid-points of the sides. A separate program was written to provide input to the NRC strip computation program whenever photographs with side fiducials are used. This program transforms the origin of the plate coordinates to the principal point and applies linear film distortion compensation to each photograph of the flight strip.

The choice of a particular method of analytic aerial triangulation depends on (1) personal preferences of the user for specific program features, (2) design of aerial camera(s), and (3) size and speed of the available computer. The NRC strip triangulation program requires less computer storage and runs more efficiently than the CGS strip coordinate computation.

For all practical purposes, either the CGS or the NRC method of analytic aerial triangulation will provide acceptable results for highway location and design purposes.

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NEW PUBLICATIONS

The Bureau of Public Roads has recently published four documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Highway Condemnation Law and Litigation in the United States

Highway Condemnation Law and Litigation in the United States contains a review of highway condemnation law as reflected in the statutes of the States. This publication was prepared for the Bureau of Public Roads by the University of Wisconsin. It consists of two volumes:

Vol. 1—A Survey and Critique (70 cents a copy). The survey was conducted on the assumption that there is too much litigation in highway land acquisition programs. One objective of the survey was to determine whether there is a close relation between the percentage of increase of contested highway condemnation cases and the seemingly higher awards that are being made by the courts as compared with appraised valuations.

The author has reviewed the law of condemnation throughout the United States to pinpoint the principal causes of litigation. The report provides a general review of condemnation law, identifies the issues that have arisen and the frequency with which they have arisen. It contains tables of highway and nonhighway condemnation cases, classification of types of proceedings involved, issues raised, and the record of success of parties on appeal.

The statutory authority to condemn property for highway purposes is reviewed, and examples are furnished to indicate the range of issues with which such statutes deal among different States. The subject of compensability is presented under three major categories—specific rights or interests, requirement of taking, and consequential damages. A description of the issues relating to procedure or practice and a review of the constitutional provisions dealing with the subject of eminent domain are provided.

Vol. 2—State by State Statistical Summary of Reported Highway Comdennation Cases from 1946 through 1961 ($1.75 a copy)—a supplement to the first volume—is a review of 1,890 reported court decisions in the highway condemnation field during the period 1946–61. It contains a State-by-State breakdown of the cases and provides supporting data for some of the findings in the basic report.

The Role of Third Structure Taxes in the Highway User Tax Family

The Role of Third Structure Taxes in the Highway User Tax Family ($2.25 a copy), a 331-page research and development report prepared under contract with the University of Mississippi, increases the factual basis for assessing the place of so-called *third structure* taxes in modern State tax systems for the support of highways. The information it provides is based on personal interviews with tax administrators and with representatives of trucking associations and individual trucking firms, on mailed questionnaires, and on examination of tax laws and public records.

The principal emphasis of the report is on the administrative aspects of third structure taxes. Its series of detailed State case studies representing 10 major types of such taxes should be particularly valuable to State legislatures, administrators, and others concerned with the problems of financing highways.

Handbook of Highway Safety Design and Operating Practices

The *Handbook of Highway Safety Design and Operating Practices* (40 cents a copy) is intended to serve all jurisdictions of government and designed to attract the attention of administrative and technical personnel who are making decisions bearing on the safety aspects of street and highway design and operations.

The publication is an illustrated guidebook presenting some of the latest safety techniques in such categories as the roadway cross-section and slopes, bridge design, signing, guardrail and barriers, drainage, and railroad grade crossings. Future supplements to the handbook will include other aspects of safety design and operations—construction and maintenance zones, urban streets and highways, light standards, gores, protective screening, etc. The handbook has a loose-leaf format to accommodate revisions and additions as new techniques are developed. Users of the handbook are urged to forward to the Federal Highway Administration, Washington, D.C., any ideas, comments, or new techniques that are appropriate for use in the handbook. The principal responsibility for the continuation of the publication has been assigned to the Bureau of Public Roads.

Standard Plans for Highway Bridges, Vol. 1, Concrete Superstructures, 1968

Standard Plans for Highway Bridges, Vol. 1, Concrete Superstructures, 1968 ($1.25 a copy), is a revised edition of the 1962 publication and pertains to bridge widths and current design specifications and geometrics. The plans are intended to serve as useful guides in the development of suitable and economical bridge designs. Sufficient information has been included so that all plans can be readily modified when contract drawings are prepared.

The volume contains plans for reinforced concrete and prestressed concrete superstructures. Reinforced concrete types include precast channel sections from 20 to 30 feet and cast-in-place I-beam and box girder spans from 40 to 120 feet. The precast-prestressed concrete types include voided sections from 25 to 40 feet, box sections from 40 to 70 feet, and I-beam sections from 35 to 90 feet. For optimum economy, the pretensioned I-beam sections have been designed to use the new 270 grade prestressing strands.

Bridge roadway widths used are 28 feet with H15–44 live load for low-traffic volume, low-design-speed roadways and 44 feet with HS20–44 live load for the standard 2-lane, two-directional roadway.

APPALACHIAN DEVELOPMENT HIGHWAY SYSTEM

STATUS OF IMPROVEMENT AS OF SEPTEMBER 30, 1968

STATE	TOTAL DESIGNATED SYSTEM MILEAGE	OPEN TO TRAFFIC		
		ADEQUATE SEGMENTS- NO APPALACHIA FUNDS EXPENDED	INADEQUATE SEGMENTS- IMPROVED WITH APPALACHIA FUNDS	TOTAL
GEORGIA	89.0	2.6	0	2.6
KENTUCKY	579.6	163.3	40.2	203.5
MARYLAND	82.2	4.1	9.4	13.5
NEW YORK	260.0	29.5	- 0	29.5
NORTH CAROLINA	199.4	1.3	14.0	15.3
OHIO	295.[illegible]	93.0	0	93.0
PENNSYLVANIA	489.9	58.1	3.1	61.2
TENNESSEE	333.3	12.8	8.6	21.4
VIRGINIA	203.8	25.0	18.3	43.3
WEST VIRGINIA	421.7	9.9	6.8	16.7
TOTAL	2,954.2	399.6	100.4	500.0

13.5%

3.4%

OPEN TO TRAFFIC
16.9%

STATE	WORK IN PROGRESS				NO LOCATION WORK UNDERTAKEN
	UNDER CONSTRUCTION	ENGINEERING AND RIGHT-OF-WAY	ROUTE LOCATION STUDIES UNDERWAY OR COMPLETED	TOTAL UNDERWAY	
GEORGIA	14.2	72.2	-	86.4	-
KENTUCKY	63.4	257.0	55.7	376.1	-
MARYLAND	3.6	37.6	27.5	68.7	-
NEW YORK	39.9	160.1	10.5	210.5	20.0
NORTH CAROLINA	28.2	118.6	26.3	173.1	11.0
OHIO	29.1	146.0	24.6	199.7	2.6
PENNSYLVANIA	45.3	146.7	236.7	428.7	-
TENNESSEE	39.9	168.8	72.6	281.3	30.6
VIRGINIA	63.7	15.5	81.3	160.5	-
WEST VIRGINIA	34.7	220.5	149.8	405.0	-
TOTAL	362.0	1,343.0	685.0	2,390.0	64.2

12.2%

45.5%

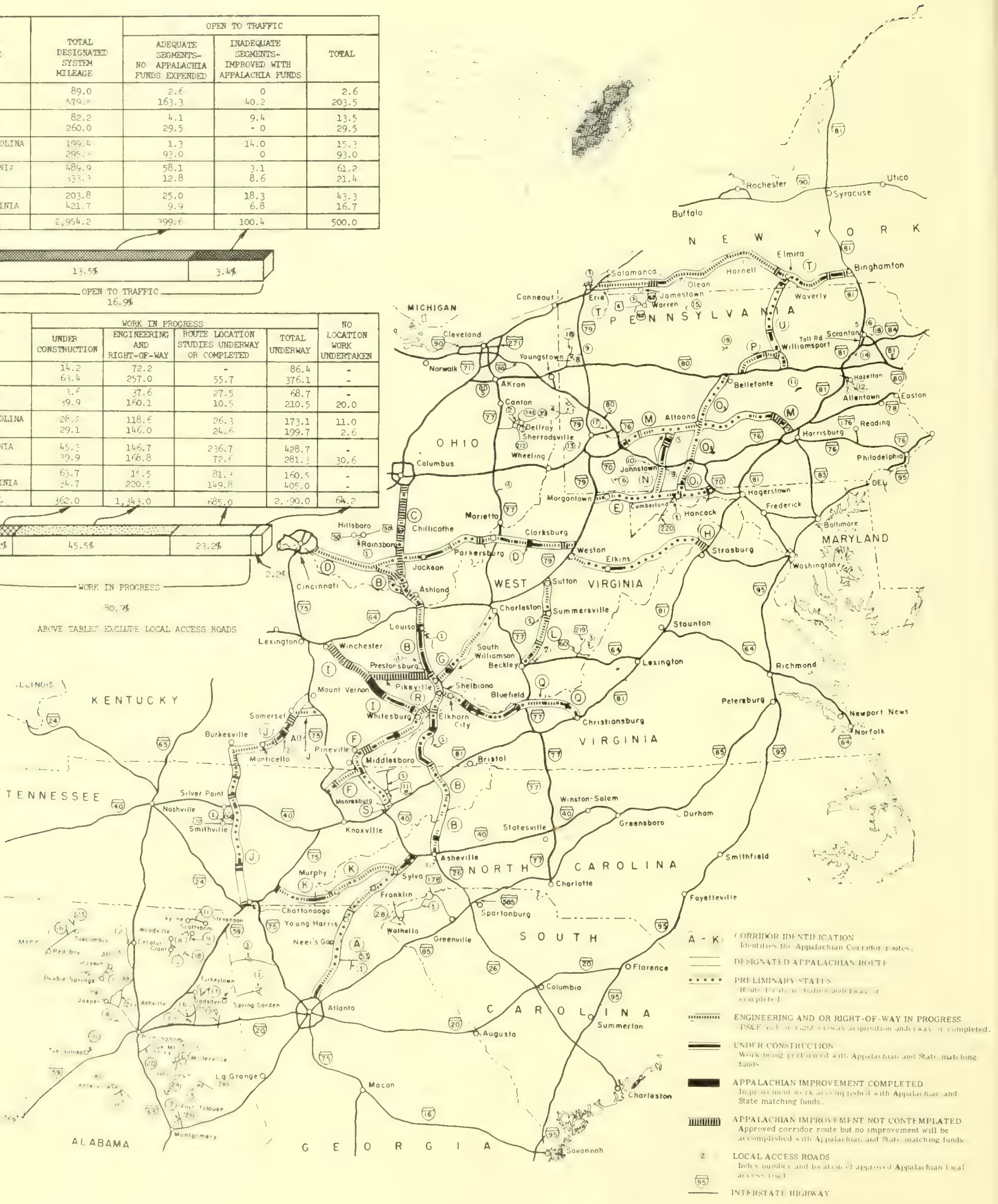
23.2%

2.2%

WORK IN PROGRESS

80.9%

ABOVE TABLES EXCLUDE LOCAL ACCESS ROADS



A - K CORRIDOR IDENTIFICATION
Identifies the Appalachian Corridor routes.

DESIGNATED APPALACHIAN ROUTE

PRELIMINARY STATUS
Route location studies underway or completed

ENGINEERING AND OR RIGHT-OF-WAY IN PROGRESS
PS&E and or right-of-way acquisition underway or completed.

UNDER CONSTRUCTION
Work being performed with Appalachian and State matching funds

APPALACHIAN IMPROVEMENT COMPLETED
Improvement work accomplished with Appalachian and State matching funds.

APPALACHIAN IMPROVEMENT NOT CONTEMPLATED
Approved corridor route but no improvement will be accomplished with Appalachian and State matching funds

2 LOCAL ACCESS ROADS
Index number and location of approved Appalachian local access road

95 INTERSTATE HIGHWAY

U.S. GOVERNMENT PRINTING OFFICE: 196[illegible]

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in Public Roads *and title sheets for volumes 24–34 are available upon request addressed to Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. 20591.*

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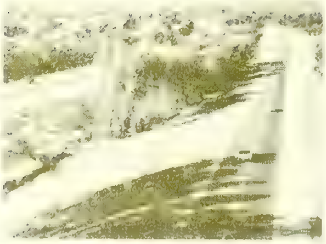

COVER

Interstate Highway 75 in Michigan between Vanderbilt and Indian River part of the Ohio-to-Soo Freeway.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 16, 1966.

Contents of this publication may be reprinted. Mention of source is requested.

Quality Assurance in Highway Construction

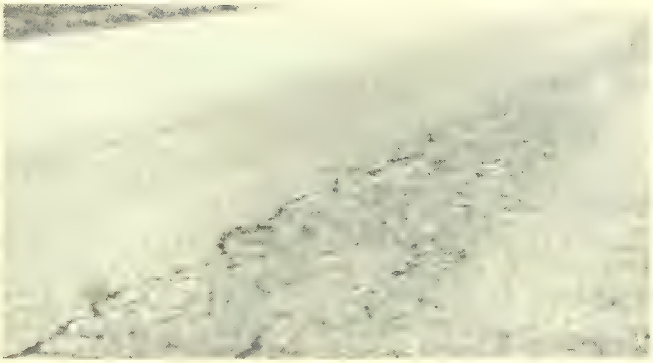
BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Part 1—
Introduction and Concepts

Reported by THURMUL F. McMAHON,
Principal Quality-Assurance Research Engineer,
and WOODROW J. HALSTEAD,
Chief, Materials Division

Beginning with this issue and continuing in several succeeding issues, the Public Roads Research staff will present an interpretative summary of the progress in its research program for the statistical approach to quality assurance in highway construction. This presentation will consist of the following six parts: 1.—Introduction and Concepts (in this issue), 2.—Quality Assurance of Embankments and Base Courses, 3.—Quality Assurance of Portland Cement Concrete, 4.—Variations of Bituminous Construction, 5.—Summary of Research for Quality Assurance of Aggregate, and 6.—Control Charts.

Statistically based quality-control methods have been used successfully in industry, particularly in the defense program, for many years. According to research results, statistical quality-assurance methods also should be adaptable to highway construction, provided that governing specifications are properly written and sampling and testing variations established to conform to the conditions of the locality in which they will be applied.



The disintegrated bituminous pavement shown above and patched concrete pavement shown below are examples of failures that may result from improper control of construction rather than from poor design.

QUALITY assurance in its broad application relates to the overall problem of obtaining the quality of construction necessary for a product to perform the functions intended. It encompasses design, production, sampling, testing, and decision criteria.

The quality of the highway product has always been a major concern to highway engineers and contractors. Traditionally, quality has been attained primarily through skills of individual engineers. When such skills are properly applied, satisfactory highway quality is obtained. However, as the speed of construction and the volume of materials to be handled increased, the traditional system became subject to breakdown. Breakdown occurs when the speed of testing does not keep pace with the speed of construction. Additionally, engineering duties have increased to the extent that engineers must spread their talents over broad areas, and many quality assurance activities have been delegated to those whose skills and experience are often inadequate for on-the-spot judgments. Moreover, legal requirements for documented evidence of specification compliance create problems.

As the Interstate program moved into its full construction phase, it became evident that the traditional quality assurance procedures were subject to criticism and that new concepts were needed. Accordingly, in 1963, the Public Roads Director of Research and Development appointed a task force to study the problem and develop a cooperative State-Public Roads research effort to improve quality assurance methods in highway construction.

The discussions and data presented here are an interpretative summary of the research progress in this area; some of the discussions already have been released by the Office of Research and Development (*1*).[1] The reader should be aware that this article pertains to a Research and Development program—not to Public Roads policy. All the proposals presented will be carefully evaluated and only those proven to be workable under actual highway-construction conditions will be adopted as parts of State or Public Roads specifications and policy.

Basic Problems of Quality Assurance

Reduced to its simplest terms, quality assurance of highway construction requires proper answers to the following three questions: (1) What do we want? (2) How do we order it? (3) How do we determine that we got what we wanted?

Answers to the first question encompass the total body of research, development, engineering technology, and experience. All these combine to define needs with respect to materials, properties, and design characteristics of the highway component.

Answers to the second question depend on the manner in which the details are spelled out in specifications—specific characteristics that must be controlled, needs with respect to qualitative level, and uniformity of the product from item to item.

Answers to the third question depend on the precision and accuracy of test methods as well as on the time required to perform the tests. Testing time often controls the number of measurements that can be made available for use in decisionmaking. More importantly, the relation of the characteristic, or property, measured by the test to the service performance of the completed component is a major consideration, which often is known only empirically, if at all.

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[1] Italic numbers in parentheses identify the references listed on p. 134.

Traditional Quality Assurance

Many specifications used today in highway construction are, in fact, recipes rather than specifications. They spell out in detail the operations of the contractor, the equipment he must use, and the desired end product he must produce. These traditional specifications have come about because adequate quality definitions and test methods pertaining to quality of the end product are lacking. When specifications do attempt to define required quality, the specified values for characteristics are often those obtained through judgment and experience. Tolerances for such characteristics seldom reflect the true needs and capabilities of the construction process or of the available materials.

When traditional specifications are combined with the skills of engineers, the complete cooperation of contractors, and the desire of everyone to do a good job, there is no doubt that a good highway can be built. However, inspectors and engineers must be capable of recognizing good materials and construction, without relying solely on quality measurements. Under most of the present procedures, one periodic sample is taken. This sample—assumed to be representative of the material or construction—is tested, and the test result is recorded as the value of the measured property, or characteristic. If the test result is within the stated tolerances, the material *passes* and is accepted. If the test result is not within the stated tolerances, the material or construction fails to *pass*. Engineering judgment must then be applied and a decision made as to whether the material should be retested or whether it may be said to *substantially comply* because the specification deviation will cause little impairment of performance.

Even though a quality assurance system that is based on engineering judgment is workable under proper conditions, the practice is difficult to define in legal or contractual terms. *Substantial compliance* has not been quantitatively defined, and the degree of acceptable variation will differ from engineer to engineer and from job to job.

To further complicate the problem, sampling and testing errors are often so large that the true variations of the materials or construction may be obscured. Some tests may not truly measure quality of the finished highway.

Improvement in quality assurance of highway construction accordingly entails:

- Development of realistic quality criteria.
- Development of valid quality tests.
- Development of valid decisionmaking rules.
- Quantification of substantial compliance.

New Developments in Quality Assurance Procedures

Statistical concepts are the most promising tools for the solution of many quality-assurance problems in highway construction. Other industries have been using statistical concepts in process control and acceptance. In fact, much of the development in this field was pioneered by the Department of Defense in its procurement program during World War II. Because of the nature of the highway industry, some of the methods must be modified, but the concepts are basic to any industry.

The science of statistics is a versatile tool. In situations requiring decisions concerning contractual items that are based on samples, statistical concepts allow varied acceptable solutions. Rules for each decision must be carefully defined and followed, but different rules can be formulated for each of the many conditions encountered. Decisions can be made with an established degree of confidence. The degree of confidence required for each decision can be correlated with the criticalness of the decision to the quality of the end product, and the rules formulated accordingly.

Test methods are continually being developed for better and more rapid measurement of quality. The greatest advance in new methods of testing has been in the nuclear field. The nuclear moisture-density gage (figs. 1 and 2) has been proved to be a fast, accurate method of measuring the moisture and density of compacted materials. Nuclear methods (fig. 3) of measuring density and asphalt content of bituminous pavements are showing considerable promise. Seismic methods of measuring compaction are also being developed. Sonic equipment is being used to test welds, and sonic methods of measuring the moduli of concrete have been in use for several years, but have not been widely accepted. Electronic equipment, using the principles of resistivity and magnetism, has been developed to check the placement of steel in concrete and to measure the thickness of pavement components.

Rapid nondestructive tests such as those cited will provide better quality control and make quality measurements available in the future.

Through the work of its different committees, the American Society of Testing & Materials (ASTM) is advancing the state of the art of quality measurement by developing precision statements for standard tests. These statements will provide a basis for evaluating the work of inspectors and laboratory technicians and should decrease testing errors.

Other aids to better quality products are automated processing plants with direct output printout. These plants provide not only automatic control, but also adequate documentation to check output for pay quantities. However, automated control is no guarantee of a quality product. One must know what to control and how precise the control must be before the benefits of automation can be attained. One area in which automation is producing dramatic results is that of surface-variation control. The *Stringline* (fig. 4), a wire guidance system to control vertical variations in concrete placement, and other guidance methods (fig. 5) have greatly improved the riding quality of pavements.

Figure 1.—The nuclear ROADLOGGER used for moisture-density determinations in compacted embankments.

Figure 2.—Moisture-density determination in compacted embankment using portable nuclear gage.

Figure 3.—Portable nuclear gage in field test to determine density of bituminous base.

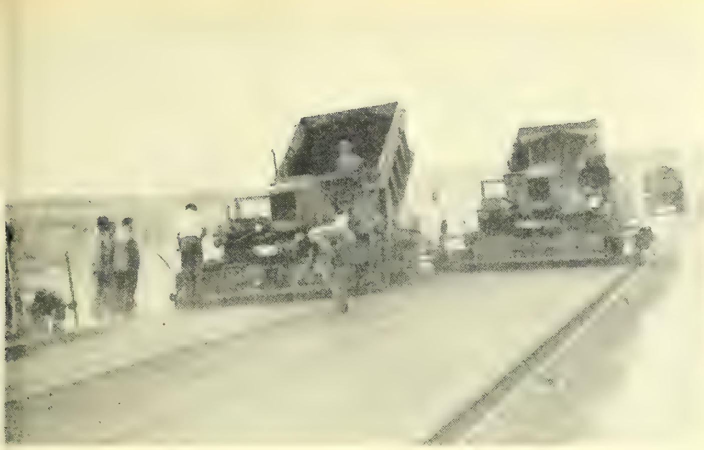

Figure 4.—STRINGLINE wire guidance system for controlling the placement of bituminous material.

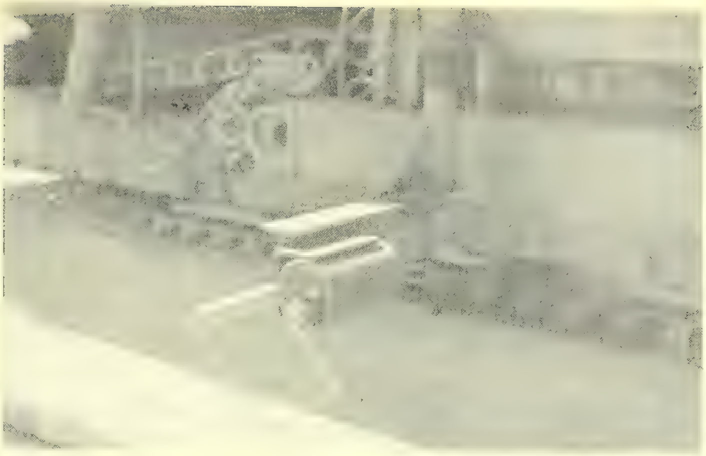

Figure 5.—SKI and wire guidance control used to provide smooth placement of pavement.

Advantages of Statistical Concepts

One significant problem in quality assurance is that of communication. Definite instructions concerning the materials and construction desired, methods to be used for determining compliance, and conditions under which payment will be made, must be given to contractors. These instructions must be explicit so that contractors, engineers, lawyers, and auditors can interpret them in only one way. The Office of Research and Development, Bureau of Public Roads, recommends that statistical concepts be incorporated in the specifications for highway construction to improve communications.

The proper use of statistical concepts will provide the following requisites:

- Statement of concise quality requirements.
- Development of valid tolerances based on the capabilities of process, sampling, and testing methods.
- Delineation of responsibility for process control and acceptance.
- Development of valid sampling plans as a basis for decisionmaking.
- Establishment of precise decision criteria.
- Development of valid proportional-payment schedules.

Stating quality requirements

In the writing of specifications, statistical concepts can be used to express quality requirements as target values for which contractors are to aim, and to specify compliance requirements as plus and minus tolerances. Tolerances from the target value, prescribed by design needs, can be based on statistical analyses of the variations in materials, processes, sampling, and testing existing in current construction practices. Such tolerances are realistic and enforceable. They take into account all the normal causes of variation and allow for the expected distribution of test results about the mean. Provisions can be made both for control to the stated level and for control of the variation from this level.

Research by the States is being undertaken to define realistic tolerances on quality requirements. From this research, it is known that test measurements on characteristics of highway materials or construction form a definite pattern grouping around a central value called the mean. The grouping indicates that test measurements in highway construction can be described in the same terms as test measurements in other industries. The measurements group around the central value in a symmetrical pattern, thereby allowing the use of statistics based on the familiar bell-shaped *normal curve.* Although some slight variation from the symmetrical curve may occur, especially when the number of test results is small, the error in assuming normal distribution of population measurements usually will not be large. If the curve is decidedly asymmetrical, skewed to the right or left, then something other than normal distribution theory must be used in the analysis.

Even though curves are normal, they may not look alike. Those with a small standard deviation will be tall and narrow, whereas

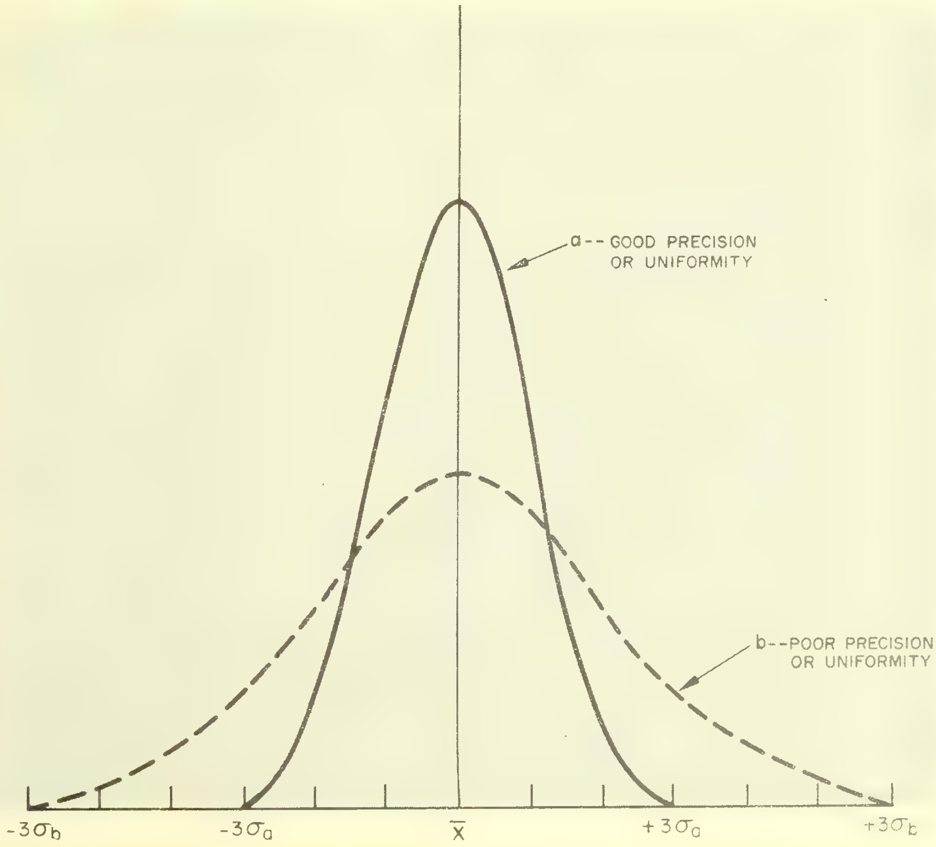

Figure 6.—Normal distribution curves.

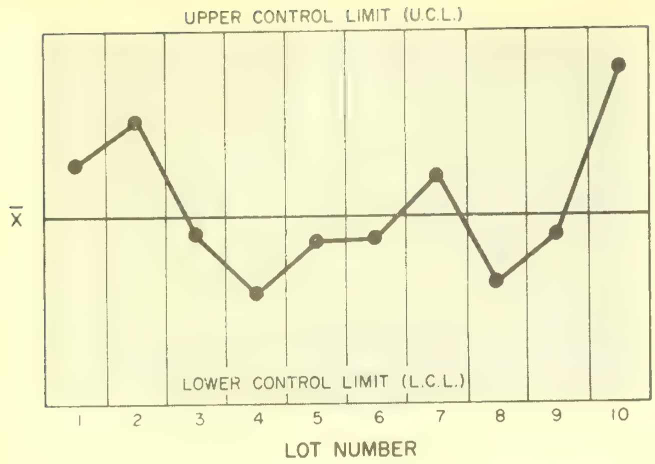

Figure 7.—Average, $\overline{X}$ control chart for n samples per lot.

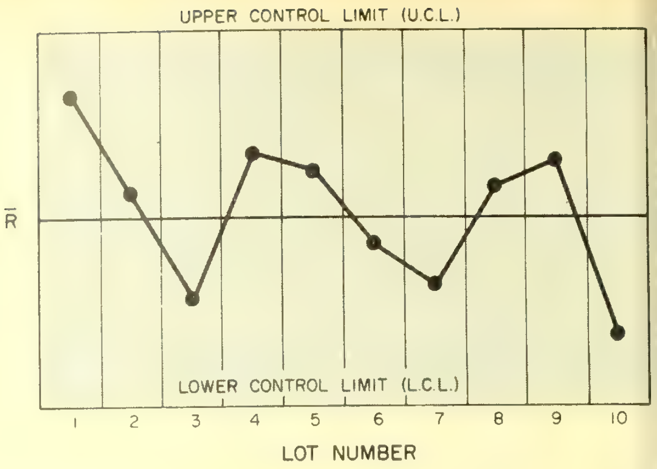

Figure 8.—Range, R, control chart for n samples per lot.

those with a large standard deviation will be short and broad. (See fig. 6.) The tall narrow curve indicates good product uniformity or measurement precision; the short broad curve indicates poor uniformity or precision.

The assumption of a normal distribution when warranted, permits the use of estimated relationships of mean and standard deviation to establish realistic specification tolerances for selected sample sizes. Such tolerances can be established by statistical analysis, together with engineering judgment, according to the degree of control needed for permissible construction risks and the economics of testing. The number of test results on which the compliance decision is based directly influences the latitude that must be given to the contractor. Often, because of the small number of tests that can be made economically, the tolerances must be wider than would seem desirable.

These relations may be stated as follows:

$$T_s = \frac{Z}{\sqrt{n}}$$

Where

T_s is the tolerance to be allowed on each side of the target value.

Z is a standardized factor equal to $(X-\overline{X})/\sigma$ that relates to the area under the normal curve for the desired confidence of decision.

n is the number of tests to be made (sample size).

Statistical concepts for quality assurance of highway construction are based on the laws of probability; consequently, these laws must be allowed to function. One of the most important requirements for proper functioning is that the data be selected by random sampling. A true random sample is one in which all parts of the whole have an equal chance of being chosen for the sample. A table of random numbers is the best device for achieving a strictly random sample, but another method of chance, such as dice, the tossing of several coins, or a wheel of chance, often will suffice in highway work. The principal requirement is that the sample not be biased by a set selection pattern or by an inspector seeking either good, bad, or representative parts for sampling.

In addition to the laws of probability, another concept, *lots*, is essential to the proper application of statistics to quality control and acceptance sampling of highway construction. A *lot* is a uniquely identified, homogeneous portion of material or construction about which a decision is to be made. The size of the *lot* may vary depending on the economics of rejection and on sampling and testing costs. The lot size must not impose a severe hardship on the contractor who encounters a rejection—the smaller the *lot* the better the contractor's position. However, small *lots* entail more sampling and testing by the State—the larger the *lot* the better the State's position. Therefore, *lot* size must be a compromise equitable to both.

Production quality control

The application of statistical concepts to highway construction allows a definite assignment of responsibility for product quality. The contractor strictly is responsible for providing quality materials and construction; the State has the prerogative of acceptance sampling and testing.

Each contractor or supplier should have a statistical quality control program that will assure his meeting the acceptance requirements of the State. Such a quality control program can be patterned after the control currently exercised by the State or it can be considerably different. Much of the control of materials and construction can be accomplished by tests, usually called indicator tests, that are somewhat simplified. These tests are less precise but more rapid than the standard tests. When proper correlation has been established, a sufficient number of indicator-test results will provide control that is as good as fewer results from more precise tests.

Control charts are among the most useful tools in production quality control. These charts, on which test results are plotted, are simple line graphs of the required quality level and of the allowable variations from this level. They pictorially present data so that everyone concerned can see the results and readily observe trends that may affect quality.

Control charts depict data in several ways, and they can be of a simple design in which the target value is used as the axis and the specification limits as the control limits. Such charts show the variation of individual values or averages with respect to the actual specifications. However, when the mean, standard deviation, and the range of the material or process can be computed from a sample, average and range charts should be used.

The average, $\overline{X}$, chart shows variations in the averages of test results. A central line and upper and lower control-limit lines are used. The range, R, chart shows variations in the ranges of test results. It also has a central line and upper and lower control limits. Construction of these charts is described in any good quality control text.

If the average, $\overline{X}$, chart is being used to control current production, a sample of n items is taken from the process at random intervals and a quality measurement made on each item. The average of these measurements is then computed and plotted on the chart. As long as the sample averages neither fall outside the control limits nor show any nonrandom variation within the limits, the process is deemed to be in control with respect to its central tendency or *target value*.

When a range, R, chart is being used to control current output, the range of a sample of n items is computed and plotted on the R chart. If the sample ranges neither fall outside the control limits nor show any nonrandom variation within the limits, the process is considered to be in control with respect to its variability. The $\overline{X}$ and R charts must be used together to assure control of both level and variation of quality. Examples of Average, $\overline{X}$, and Range, R, charts are shown in figures 7 and 8.

Acceptance procedures

For highway construction, the State may elect to use the results of supplier's or contractor's quality-control programs to accept material or construction. However, the usual procedure in buyer-seller relations is for the buyer to establish independent acceptance plans for each item of material or construction. An acceptance plan designates lot size; where and when to sample, on a random basis; numbers of samples to be taken; method of test to be used in the quality measurement on each specified characteristic of the sample; and, based on the test results, procedure for making a decision. An acceptance plan may be a simple statement or a complicated system in which many steps must be taken before a decision can be made. Examples of sampling plans will be included in subsequent installments of this article.

When decisions are based on a sample, a basic truth must be accepted: There is a certain risk that the decision is incorrect because the sample does not truly describe the total of the material. One advantage of the statistical approach is the ability to design a sampling plan in which the probabilities of acceptance of poor material, the β risk, and the rejection of good material, the α risk, are known. When *good* and *bad* materials have been defined and the risks to be taken agreed upon, the number of samples required to make a decision compatible with the risk probabilities can be calculated. These relations and the methods for establishing an operating characteristic curve, which denotes the probability of acceptance for intervening qualities of a product, can be found in any good quality control text.

Summary of Research Effort

During the past 4 years, the Office of Research and Development, Bureau of Public Roads, has actively promoted the following five-point program of research in quality assurance in highway construction:

- Awakening the highway industry's interest in the utility of the statistical approach to quality control and acceptance testing.
- Developing guides for research that would yield statistical data for writing acceptance specifications.
- Planning and coordinating a nationwide program of research in applying statistical methods to highway construction.
- Gathering and analyzing data and disseminating research findings.
- Designing and implementing projects by which the findings of the research program can be evaluated.

This effort is basically State research financed with Highway Planning and Research (H.P. & R.) funds. Many of the studies have been conducted according to guidelines established by Public Roads Task Force; others have followed plans developed by State personnel.

Early in the research program it was realized that little data were available for use in establishing quality levels and variations in highway construction on a statistically valid basis. Therefore, a concerted effort was initiated to measure quality and its variations in terms of existing criteria. Participating State highway departments have been measuring the level and variability of quality in their construction. To date, 28 States have conducted studies funded under H.P. & R. contracts and seven others have been investigating construction in State-funded studies.

The objective in the formulation of all studies was to produce compatible information that could be used throughout the Nation. A booklet of guidelines (*2*) was prepared and distributed to the States for use in planning their projects. A method of obtaining statistically valid data for an analysis of variance to isolate the components of variance was outlined in a suggested research plan. The plan permits overall variance to be divided into material or process variance, sampling variance, and testing variance.

According to the research data received from the States, 50 percent or more of the overall variance could be attributed to sampling and testing in some of the studies. Results showing this magnitude of sampling and testing error indicate that a concerted effort should be exerted by each highway department to train inspectors and laboratory technicians.

Also, according to the research data, which has been statistically analyzed to determine the percentage of present construction that complied with the levels and limits of current specifications, a considerable portion of the construction is shown to be outside the limits defined by the specifications. In fact, as much as 30 percent of some construction, considered to be completely acceptable under current control procedures, may be outside the stated limits. This variation from the specifications, in part, reflects the errors of sampling and testing, but there are indications that many of the present limits do not reflect valid allowances for the variable materials and processes used in highway construction.

Supplementing the State research effort, the Public Roads Office of Research and Development began a contract-research program in 1963 to further the development of statistical quality-control applications to highway construction. Many aspects of the task force's research plan were based on the results of the initial study in which the contractor evaluated the choice of concepts available and pointed to the priority areas for study. The study conclusions were presented in an unpublished report entitled *A Plan for Expediting the Use of Statistical Concepts in Highway Acceptance Specifications*. Two subsequent contracts provided valuable information concerning the level and variation of quality in base and subgrade construction.

A review of the Public Roads Standard Specifications for *Construction of Roads and Bridges on Federal Highway Projects* (FP–61) was conducted by another contractor. The final report on the contract was later used to develop a *futurized* revision of FP–61—the first attempt at writing complete specifications using statistical concepts wherever feasible.

The *Futurized Revision of Federal Project Specifications* was never intended for use in highway construction, and distribution of the document has not been widespread. However, it has been reviewed by many outstanding highway engineers and by committees of the American Road Builders Association (ARBA) and other organizations. Most of the comments received have been favorable to the concepts incorporated in the sp[illegible] some disagree with methods of accomplishment and with items other than those that were treated statistically. The statistical applications embodied in the *Futurized Revision of Federal Project Specifications* have been proved to be sound and are [illegible] many specifications now being written.

Subsequent information obtained from the States' research studies and Public Roads' in-house research has been used in the development of statistically based research specifications for construction of embankments, bases, and bituminous pavements. These specifications have been studied and discussed by many engineers associated with highway construction. It is evident from the comments received that some of the ideas presented are still not completely acceptable to the industry. Objection has been voiced to the complete delegation of quality control responsibility to the contractor and to the reduced payment schedules for nonconforming materials and construction. Primarily, the differences of opinion concern the degree of responsibility and the amount of reduced payment.

Undoubtedly, changes in present contractor-State relations are needed to fully implement the statistical approach to specifications. These changes must establish end-result requirements that can be measured by the States. Practical considerations such as inadequately trained manpower, equipment availability, and lack of adequate end-result tests in some instances prevent an immediate, complete changeover from the *traditional* specifications. However, a number of States already are assessing the degree to which they are involved in the process control and are shifting as much of the responsibility to the contractor that is possible under present circumstances. Where adequate tests to measure finished quality are available, there is no evidence that ultimate responsibility for process quality would present a hardship to the contractor. Increased contractor responsibility coupled with proper flexibility by the State should result in better and more economical construction and provide incentive for the equipment industry to produce equipment that is capable of high-quality work as well as high production.

For certain operations, reduced-payment schedules for out-of-limits construction seems to be a necessity. The designation of really *good* material or construction and really *bad* material or construction is relatively simple. However, there is usually a grey area in which the out-of-limit material or construction may be usable, and removal and replacement operations are not warranted because of delays or other hindrances to traffic. For such material or construction the concept of partial payment is not new. In current practice, payment to the contractor is arbitrated in after-the-fact negotiations. If schedules are established before the contract is let, the contractor will be aware of the risks involved and after-the-fact penalties probably will not be necessary.

Although objections have been raised to some concepts advocated in the research program, the basic idea of adapting statistical concepts to highway construction is being well received. Research data are being used by many States to revise specification limits to allow for sampling and testing errors determined through the research studies. Only one State has progressed sufficiently to include a complete statistical approach in its standard specifications. At least five States are known to be incorporating special provisions that were calculated on a statistical basis. Five other States have written statistically based specifications for some facet of their construction, but have not used them in contractual work.

Rapid progress is being made in the adoption of control charts for displaying and analyzing data. Control charts can be used under present specifications if the inherent limitations are well understood. Their use will be greatly enhanced as more information on quality requirements and measuring techniques are developed.

Optimum use of statistics in quality assurance can come only through the adoption of end-result specifications. End-result specifications will allow the proper designation of responsibility for control and acceptance, and they are the only means through which quality measurement of a completed segment of construction will ever evolve. End-result specifications require knowledge of end requirements and must be based on measurements made on the end product. The highway industry's present inability to adequately define performance requirements and to measure performance quality dictates a major redirection of the research program.

Discussion of Research Results

Information, data, and analyses obtained through research by the States, Public Roads, and others are presented in subsequent parts of this report. These data provide support for many of the statements in this introductory section.

The indicated variation in materials and construction is, in fact, often attributable to variation in sampling and testing rather than to the materials or the construction itself. It is essential, therefore, that each State determine the sampling and testing variation associated with its current methods and personnel, and that it make a concerted effort to reduce test variations to a minimum.

Many current specifications do not adequately allow for sampling and testing variations in the presently prescribed limits. Where such inadequacy exists, and it is impossible or uneconomical to further reduce these variations either by improving the procedure or increasing the number of tests, the specification limits should be relaxed.

When random samples are taken in sufficient number to adequately measure quality, it has been shown that a surprisingly large portion of currently acceptable construction does not comply with present limits. It may not be economically feasible to make sufficient tests for accurate measurement of quality during the control and acceptance process, but as stated earlier, the use of statistical concepts will make it possible to select the sample size in accordance with the importance of the decision being made and the economics of sampling and testing. It is therefore important that the validity of current tests as indicators of quality be studied, and that new tests be developed that will better measure the performance of the end product.

The variability of materials and construction is emphasized by the data. Present procedures usually are concerned with the average level of characteristics; however, even when the target value is met, it is shown by statistical analyses that a large portion of the materials or construction may be outside specification limits. Accordingly, variation, as well as the level of quality, should be controlled. To accomplish this control, a method of random selection of samples must be used. The adoption of random sampling by industry will significantly improve quality assurance in highway construction.

The research program has produced many other findings that will be discussed in subsequent sections. However, additional data are required before the results can be established as facts. Some of the data being received are not sufficiently complete to firmly establish the necessary basic relationships.

Discussions and findings for specific items of construction will be included in the next and subsequent issues.

REFERENCES

(*1*) *Quality Assurance Through Process Control and Acceptance Sampling*, reported by the Statistical Quality Control Task Group, Office of Research and Development, Bureau of Public Roads, April 1967.

(*2*) *The Statistical Approach to Quality Control in Highway Construction*, Research Guides, The Statistical Quality Control Group, Office of Research and Development, Bureau of Public Roads, April 1965.

Thermoplastic striping material on southbound lane of Atwells Avenue Bridge, Interstate Highway 95, Providence, R.I.

Comparison of the Performance and Economy of Hot-Extruded Thermoplastic Highway Striping Materials and Conventional Paint Striping

Reported by BERNARD CHAIKEN,
Principal Research Chemist,
Materials Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

In the survey reported here, the comparative durability, performance, and economy of hot-extruded thermoplastic traffic-marking materials and conventional paint striping were evaluated. All State highway departments, major toll road agencies, and several larger cities and county road authorities were included in the survey, in which it was shown that the relative durability and long-term economy of hot-melt thermoplastic striping materials were greatly affected by type of pavement, snowplow activity, and traffic density. It was also shown that, to a lesser extent, other factors also affected the relative merits of these materials. A guide chart was developed to facilitate selection of the more economical of the two marking materials. Selection parameters include pavement type, traffic density, and mean annual snowfall—snowfall being an indirect measure of potential snowplow activity. The chart permits selections on the basis of direct comparative costs alone, as well as on the basis of additional indirect costs such as traffic delays and potential accident hazards attributable to frequent conventional maintenance striping.

Hot-extruded thermoplastic was found to be more economical than traffic paint under conditions of high traffic density and limited snowplow activity; otherwise standard traffic paint was the more economical of the two methods of striping. Bituminous pavements showed the thermoplastics to better advantage than did portland cement concrete surfaces.

Introduction

FOR more than a decade, many highway agencies have been developing an interest in the use of hot-extruded (hot-melt) thermoplastic traffic striping materials as an alternative to conventional paint striping. This type of thermoplastic differs from the cold-flow preformed type, as well as from the hot-spray type, in that it is extruded onto the pavement in a molten state. The hot-extruded material is applied by first heating the solvent-free solid product to its molten state, at approximately 425° F., and then extruding the molten material directly onto the pavement through a die. The produced traffic stripe, about ⅛-inch thick, solidifies within minutes, and can be exposed to traffic much sooner than conventional paint stripes.

During 1965, the State highway departments reportedly used almost 5 million linear feet of hot-extruded thermoplastic striping, which is equivalent to about 900 actual stripe miles of the material (*1*).[1] Its growing popularity has been attributed to its rapid drying, or set, compared with traffic paint as well as to its superior durability, which thereby obviates the need for frequent stripe maintenance. Thus, compared to conventional striping the material potentially offers advantages of long-term economy and traffic safety. Major limitations to a wider use of the material have been the initial installation cost—15 times the cost of ordinary striping—and premature failures caused by loss of adhesion to the pavement surface. In reports on two surveys conducted by the author several years ago, the advantages and limitations of this material were discussed (2, 3). Several other reports on the performance and merits of such thermoplastic striping are available and still others will be published soon (4, 5, 6, 7).

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[1] Italic numbers in parentheses identify the references listed on p. 154.

In general, evidence to date has shown that:

- Thermoplastics are much more durable on bituminous pavements than on portland cement concrete pavements.
- The newer the concrete surface the poorer the adhesion; the material, after hardening, is somewhat subject to blistering, especially when applied to concrete.
- The thermoplastic is subject to snowplow damage.
- Thermoplastics may be more economical than conventional paint striping only when high traffic volumes are prevalent.

Except for these generalities, no clear-cut broad geographical criteria have ever been established to define precisely where and how such materials may be used to economic advantage. Moreover, the technology of applying thermoplastic striping has been improved in recent years; therefore, it seemed desirable to conduct an up-to-date investigation of the performance and economics of thermoplastics, and, in 1967, a survey was initiated by the Bureau of Public Roads to evaluate new developments and performance data. The objective of the survey was to develop, if possible, clear criteria on the relative long-term economics of thermoplastic and paint striping for any given location. The results of that survey are reported here.

Agencies Surveyed and Information Sought

The type of information requested in the survey is listed under the heading *Data Requested in Survey*, page 155. In requesting the information, emphasis was placed on recent installations where quantitative information and conclusions were available. The requested information was formulated to yield quantitative data on the comparative long-term economics of paint and thermoplastics, as well as to obtain criteria used in different localities to select one material over the other.

The inquiry was sent to all the State highway departments, to the highway departments of the District of Columbia and Puerto Rico, to most of the toll road agencies, and to some of the larger cities and counties. The number and type of agencies surveyed, the replies received, and the number of agencies

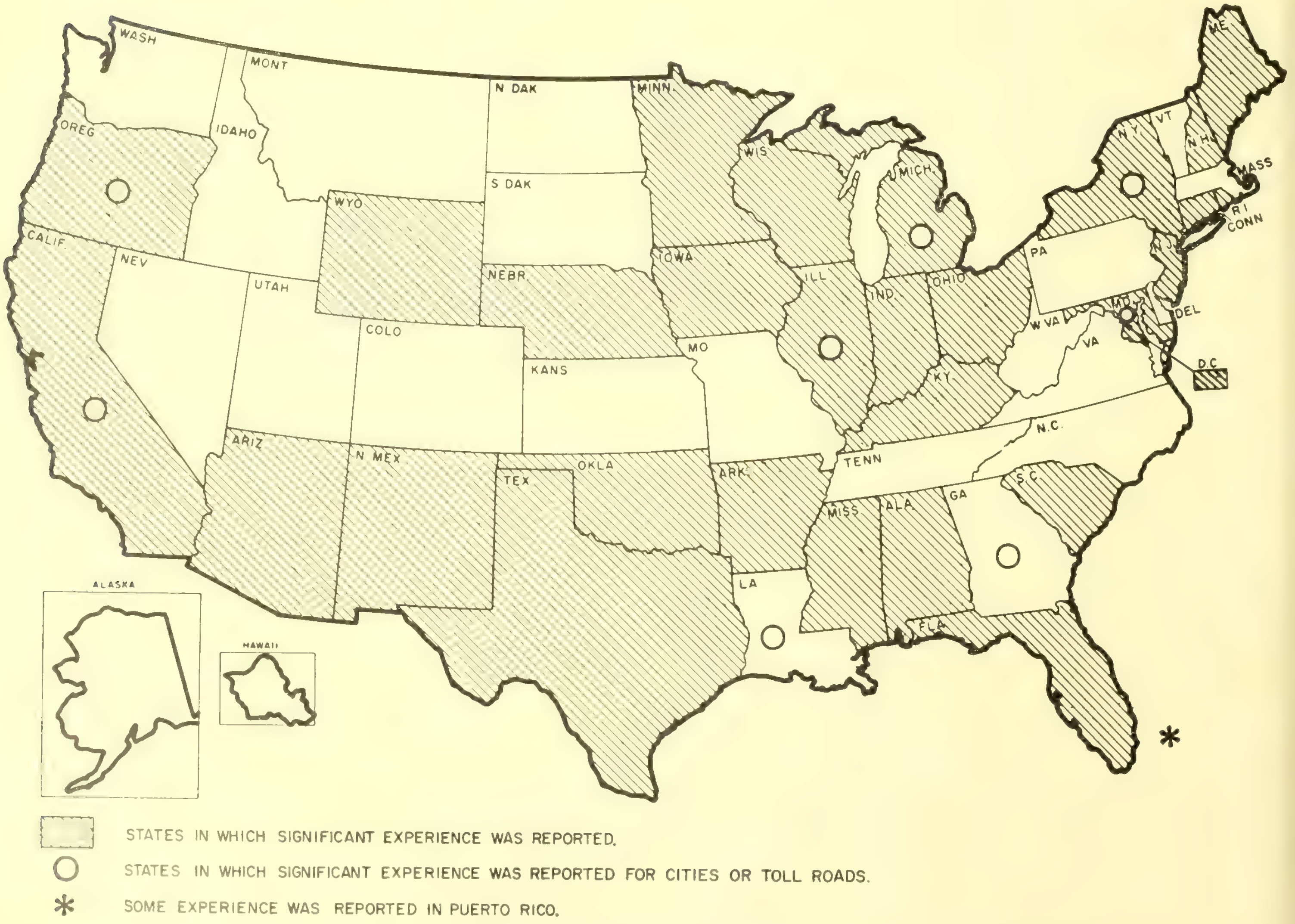

Figure 1.—States in which highway agencies reported significant experience in use of hot-extruded thermoplastic striping.

Table 1.—Summary of responses to questionnaire

Highway or bridge agencies	Number surveyed	Replies received	Number reporting significant experience with thermoplastic
States, District of Columbia, and Puerto Rico	52	48	31
Toll-road and bridge authorities	[1] 17	16	4
Cities and counties	[2] 16	13	8

[1] Included: Delaware River and Bay Authority, Florida State Turnpike Authority, Illinois State Toll Commission, Indiana Toll Road Commission, Kansas Turnpike Authority, Massachusetts Turnpike Authority, New Jersey Highway Authority (Garden State Parkway), New Jersey Turnpike Authority, New York State Thruway Authority, Ohio Turnpike Commission, Oklahoma Turnpike Authority, Pennsylvania Turnpike Commission, Port of New York Authority, Richmond-Petersburg Turnpike Authority, Triboro Bridge and Tunnel Authority, Texas Turnpike Authority, West Virginia Turnpike Commission.

[2] Included: Atlanta; Baltimore; Boston; Chicago; Dallas; Detroit; Los Angeles; Los Angeles County; Newark, New Jersey; New Orleans; New York; Philadelphia; Portland, Oregon, Providence; San Francisco; San Francisco County; and Seattle.

reporting significant experience with hot-extruded thermoplastics are summarized in table 1. Approximately half of the road agencies surveyed reported some recent significant experience with the material.

In figure 1, a map of the continental United States, the areas that reported significant recent experience with thermoplastic striping are shown; the major areas of the country were well represented by thermoplastic installations.

Tabulation of Survey Results

The responses of all the road agencies reporting substantial experience are summarized in tables 2 and 3, as well as in the section *Comments by Agencies Surveyed,* page 155.

Table 2 is a tabulation of the data received. In column R of table 2, the average useful life of the thermoplastic stripe reported by each agency is shown, either on the basis of fully achieved life or on an estimated basis wherever the stripe was still considered to be serviceable at the last inspection. Results shown in parentheses were estimated by the author when the agency failed to report the estimated life and sufficient data were supplied. These estimates were calculated from the percentage of stripe lost up to the time of the last observation and were extrapolated on the assumption that the material would reach its terminal point when 40 to 50 percent of the stripe was lost—an approximation of the terminal point criteria used by several surveyed States and of that mentioned in a separate survey conducted by the Institute of Traffic Engineers.[2]

Related data on conventional paint striping for the same or comparable locations as those used for thermoplastic striping are shown in columns S and T of table 2. The relative long-term economics of thermoplastics and paint are presented in column U. The data in column U were calculated by dividing the unit annual maintenance cost of thermoplastic striping by that for conventional paint striping. Values greater than one indicate that conventional paint striping has a long-term economic advantage over thermoplastics; values less than one indicate the reverse situation. Here again, values shown in parentheses reflect calculations based on the author's estimates. More than one numerical entry in any column of the table reflects the data for more than one type of stripe, as shown in column O. The first numerical entry is for the first type of stripe listed in column O, etc. Where a single numerical entry is shown in column U or preceding columns, and more than one type of stripe is shown in column O, the numerical entry refers to the first type of stripe listed in column O—usually a lane line.

The policies or criteria reported by the various agencies as to the conditions under which they permit or justify the use of hot-extruded thermoplastics striping are summarized in table 3. In those agencies with established policies, the use of thermoplastics is generally restricted to areas of high traffic density and excluded from locations subject to heavy snowfall. A few agencies use the material only on bituminous pavements because of its somewhat erratic performance on concrete pavement. The few agencies reporting numerical criteria in terms of average daily traffic (ADT) show a considerable variation in this requirement; this will be discussed more fully later.

A summary of the qualitative and subjective remarks by the agencies reporting experience with hot-melt thermoplastics is presented in the section *Comments by Agencies Surveyed,* page 155.

General Observations From Survey Data

Some general observations are evident from the data shown in tables 2 and 3 and in the section *Comments by Agencies Surveyed,* page 155. These observations are discussed in the following paragraphs and, unless otherwise indicated, the remarks are applicable to data on 4-inch-wide longitudinal striping—either center lines or lane lines rather than edge or transverse markings.

Installation costs of paint striping and hot-melt thermoplastics

The reported costs for conventional paint striping varied from as little as 0.9 cent to as much as 10 cents per linear foot of 4-inch stripe. All reported costs reflect the entire installation cost—materials, labor, expendable supplies, equipment depreciation, etc. The 10-cent figure, much higher than the other reported costs, was reported by authorities in New York City with the explanation that the city's high traffic-control expenses accounted for much of the cost. In general, the reported costs for conventional striping were based on installations made by the agency itself rather than by a contractor. From the data reported, the average cost of paint striping for open highways was calculated and determined to be 2.2 cents per linear foot of 4-inch longitudinal striping. This average cost seemed to compare reasonably well with a detailed cost analysis of paint striping made by one agency, the New York Department of Transportation, which reported an average cost of 1.7 cents per linear foot of paint stripe for the year 1963 after a an intensive study of this aspect alone (*8*). It was shown that approximately 36 percent of the cost, 0.6 cents, was for paint, and that the remainder, 1.1 cents, was for other items—glass beads, labor, fuel, supplies, equipment depreciation, etc. Assuming the normal inflationary rise between 1963, the time of the New York study, and the time of the survey reported here, 1967, the two cost figures are comparable. A few agencies reported contract costs for paint striping, which were approximately one and one-half to three times as high as when the agency did the striping with its own forces.

The reported costs of thermoplastic installations ranged from a low of 17 cents to a high of 63 cents a linear foot of longitudinal 4-inch stripe. Generally, the few agencies reporting extremely low costs had either performed the work themselves or stated that the contract price was the same as the contractor's cost or below it because the contractor had taken a loss to demonstrate the merits of the material. In general, extremely high costs were reported only for very small installations or for city installations in which the cost reflected expensive traffic control and slow application rates. The average cost of all 4-inch longitudinal thermoplastic striping was calculated to be 32.7 cents a linear foot, and generally represented the average contract price for large installations on open highways. This calculated cost is similar to that reported in another survey (*2*) in which it was pointed out that the installation cost is very much affected by the quantity installed and the extent of the performance guarantee provided.

Pavement precleaning prior to thermoplastic application

The type of pavement precleaning reported by the highway agencies differed. Some agencies did not preclean the pavement; others precleaned by one of the following methods: Sandblasting, brooming, air blasting, buffing, or acid etching. Many agencies did not know the exact nature of the precleaning performed. The most prevalent practice, especially on bituminous pavements, was no precleaning at all. There seemed to be no significant trend in the type of precleaning with respect to pavement type, except that sandblasting and acid etching were restricted to concrete pavements. Agencies having extensive, recent experience with thermoplastic materials prefer special precleaning methods for concrete surfaces. For example, the

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[2] *Model Specifications for Thermoplastic Pavement Marking Materials,* Institute of Traffic Engineers, Committee 7M(66), Sept. 30, 1967. (Unpublished draft of standard under review.)

Table 2.—Tabulation o

Agency	Project and location	Test-site data: ADT [2] Total	ADT [2] Per lane	Pavement Type [3]	Pavement Age [4]	Snowplow activity	Approximate mean annual snowfall [5]	Thermoplastic-striping data: Date striped	Pavement pretreatment [6]	Primer used	Name of thermoplastic [7]
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	(K)	(L)
											STATE OR
		Thousands	*Thousands*		*Years*		*Inches*	*Month-year*			
Alabama	U.S. 231, between Tennessee River and Huntsville	15-30	4-7	B	New	Negligible	4	3-59	None	None	Perm, Cat
Arizona	I-17, Phoenix	50	8.3	C	4	None	0	10-62	do	Rubber	Perm
Arkansas	I-30, S. of Little Rock	12	3	C	0.3		4	4-63		Pliobond	do
California	I-80, 5,000 ft. elevation			C, B		Heavy	100+	?-60	sbl [19]	do.[20]	Perm, Cat
	Routes 20, 49, and 149, Grass Valley, 2,500 ft. elevation.		5	B		Light [21]	60	9-65	do	do	[illegible]
	04-Ala. 24, Berkeley		8	B		None	0	12-58			
	04-SF-101, San Francisco		9	B		do	0	?-61			
	04-SF-101, Bayshore Freeway, San Francisco.		20			do	0	7-58			
	F 07 1-1-1, Los Angeles		3.5			do	0	?-64			
	I 605-2 76 127, Los Angeles		10			do	0	?-64			
	Summary and general observation	[illegible]		B	New and old	do.	0	?-60	None	None	[illegible]
		[illegible]		C	do	do.	0	?-60	sbl [20]	Pliobond	do
Connecticut	I-91, Rocky Hill to Hartford	42	7	C	New	Heavy	50	11-63	None	Epoxy [24]	Perm
Florida	Gainesville, NW 3, 5 and 7 and University Ave.	17	4	B		None	0	3-60			do
		17	4	B		do	0	3-60			Cat
	IP-506-A, Gainesville, Waldo Road.	6	1.5	B		None	0	7-63		None	
	IP-506-C	8.6	4.2	C		do	0	7-63		Unidentified.	
	I-95, Dade Co., Miami	87	11	C		do	0	3-65	abr	Epoxy	Perm
Illinois	FAI-55, Stevenson Expressway, Chicago		13	C	1	Heavy	36-60	11-65		do	do
	Edens Expressway, Chicago	90	15	C	6	do	36-60	?-57	brm	Pliobond	do
	General observations (Chicago area).	High				Heavy	36-60				
Indiana	SR 53, Gary		9	C	Old	Light	60	7-63	None	Epoxy	Perm
	U.S. 27, Fort Wayne		8.8	B	do	do	36	7-63	do	do	Cat
	U.S. 31, Franklin		4.2	B	New	do	12-24	6-64	do	do	do
Iowa	I-235, Des Moines Freeway	44	11	C	do	Moderate	24-36	6-64	Acid	do	Perm
Kansas	I-70, Topeka		2	C	1	do	12-24	1-62	brm	Rubber	do
	U.S. 50, Kansas City		7	B		do	12-24	11-59	do	None	Cat
	U.S. 69, Kansas City		4.3	B		do	12-24	7-59	do	do	Perm
Kentucky	I-264, N. U.S. 60, Louisville (Site 1)	19	5	C	1	Medium	12	11-62	None [25]	Pliobond	do
		19	5	C	1	do	12	11-62	do	do	Cat
	I-264, S. U.S. 60, Louisville (Site 2A)	55	14	B	2	High	12	11-62	do [26]	do	Perm
		55	14	B	2	do	12	11-62	do	do	Cat
	I-264, S. U.S. 60, Louisville (Site 2B).	48	12	B	2	do	12	11-62	do	do	Perm
		48	12	B	2	do	12	11-62	do	do	Cat
	I-264, U.S. 60, Louisville (Site 2C).	43	11	B	2	do	12	11-62	do	do	Perm
		43	11	B	2	do	12	11-62	do	do	Cat
	I-64, Franklin-Shelby Co. (Site 3).	10	2.5	C	1	Medium	12-24	10-62	do	do	Perm
		10	2.5	C	1	do	12-24	10-62	do	do	Cat
	I-64, Clark-Montgomery Co. (Site 4).	5.8	1.5	B	2	Low	12-24	11-62	do	do	Perm
		5.8	1.5	B	1	do	12-24	11-62	do	do	Cat
	U.S. 60, Frankfort-Versailles Road (Site 5).	9.7	2.4	C	5.5	Medium	12-24	6-65	Air	Epoxy	Perm
	I-64, S. Frankfort (Site 6)	8.8	2.2	C	3.5	do	12-24	6-65	do	do	do
	I-64, E. of I-264, E. Louisville (Site 7).	15	4	C	0.6	do	12	6-65	do	do	do
	I-65, N-S Expressway, E. Louisville (Site 8).	52	13	C	2-8	High	12	6-65	do	do	do
	I-264, SE. of Louisville (Site 9)	59	15	B	9	do	12	6-65	do	None	do
Maine	Augusta, State St.		6	B	Old	Heavy	60-100	8-66	None	do	Cat
Maryland	Baltimore Beltway	55	7	C	New	do	12-24	6-65	Unknown	Epoxy	Perm
	Md. 193, Conn. Ave.	40	10	C	Old	do	12-24	5-65	do	Unknown	Cat
		40	10	B	do	do	12-24	5-65	do	do	do
	U.S. 40, Flintstone Bypass	3.2	1.6	B	New	do	12-24	?-59	None	do	Unknown
Michigan	U.S. 127, S. of Lansing			B	Old	Heavy	36-60	?-58			do
				C	do	do	36-60	?-58			do
	I-75 & I-375, Chrysler Freeway, Detroit.	30	5	C	New [36]	Light [37]	36	6-64	sbl	Epoxy [38]	Perm
Minnesota	I-494-4(65)231, near Minneapolis	58	10	C	2-4	Heavy	36-60	9-63	Buff	Synthetic rubber.	do
		58	10	B [39]	2-4	do	36-60	9-63	do	do	do
	I-35W-3(81)112, Minn.-St. Paul	26	4	C	2	do	36-60	5-66	None	Epoxy [40]	do
		26	4	B	2	do	36-60	5-66	do	do	do
	I-35-6(82)205, near Willow River	3	1	B	1	do	60	9-66	do	do	do
Mississippi	I-55-4(23)240, N. of Batesville	5.3	1.3	C	New	Slight	2-4	9-62	Clean [41]	Pliobond [42]	Cat
		5.3	1.3	B	do	do	2-4	9-62	do	do	do
	I-059-2(9), N. of Hattiesburg, Highway 49	7	1.8	C	New	None	0	11-61	do [41]	do [42]	do
	I-059-1(8), S. of Hattiesburg, Highway 49	6	1.5	C	Old	do	0	4-62	do	do	do
	F-018-3(2), Starkville (MSU), Highway 12	2.8	0.7	C	New	do	2	12-64	do	do	do
Nebraska	I-80-4(106), near Lincoln		3.5	C	1	Moderate	24-36	?-63	None	Synthetic rubber.	do
	I-80-9(11), near Omaha		8.1	C	5	do	24-36	?-63	do	do	do
	I-280-1(8), near Omaha		5.6	C	6	do	24-36	?-63	do	do	do
New Hampshire	I-93-2(20)39, Concord to Canterbury.		3.0	B	New	Heavy	60-100	11-59	do	None	Perm
New Jersey	U.S. 1, N-S Freeway, Trenton	20	5	C	Old	Moderate	12-24	?-60		Unknown	Perm, Cat
New Mexico	I-25, Albuquerque		4.2	C	0.25 and 4	None	12	5-63	do	None	Perm

See footnotes at end of table.

Thermoplastic-striping data—Continued										Standard paint-striping data [1]					
Amount of striping		Type of stripe [8]	Installation cost per linear foot of stripe [9] (P)			Amount of stripe lost (Q)		Average useful life (R)		Materials and installation cost per linear feet [10] (S)			Average useful life	Ratio of long-term cost, thermoplastic-paint [11] (U)	
(M)	(N)	(O)	4-inch width	6-inch width	8-inch width	First year	Total lost	Actual [12]	Estimated [13]	4-inch width	6-inch width	8-inch width	(T)	Determined	Estimated [14]
TERRITORY															
Miles	*Thousands of linear feet*		*Cents*	*Cents*	*Cents*	*Percent*	*Pct./No. of yrs.*	*Years*	*Years*	*Cents*	*Cents*	*Cents*	*Years*		
28		L	40			0	8/0		[15] 12	2.5			[14] <1		<1.3
1		L	40			0	0/5		8	4			0.5		0.6
2.5		L, E, T	[16] 35		85		[17] 8/2		3	[18] 5		[18] 9	0.5		1.2
						>50	50/0.25	1/4						>1	
	14		[23] 27				1/2		4	3			0.25		0.63
1							50/9	[24] 9							
1.6						0	0/6		8						
								>8							
							20/3		5						
									6						
		L, etc.	[23] 24		[23] 5	0	0/7		>8	3		15	0.75–2	<1	
		L, etc.	do		do	5	35/7		5	3		15	do	1.6	
	132	L, E, G	32	48	64	10	? 4	>4.5	6	[27] 3	[27] 3.5	[27] 6	0.3	<1	
		L, T	24		48								0.2		
		L, T	42										do		
		T						>4					1.8		
		T						>4					0.9		
1		L	[28] 35			(29)				2			>0.2		
	670	E, L, G	[30] 25	[31] 35	56		15/2		4	[30] 0.8	[31] 4.5	5.4	0.33		1
		L						4					0.25	[32] 0.6	
		L, G		[31] 35	56		50/4	4			[31] 3.1	5.4	0.33, 0.50	1, 1.3	
	31	L	33			30	45/1.5	1.5		2			0.33	3.7	
	2.5	L	35			Minor	Minor		7	2			0.5		1.2
	11	L	31			do	do		7	2			1		2
	29	L	[33] 46		64	[34] 70	50/3		3	[33] 1.2			0.5		(6)
	9	L	39	55	72		[34] 25/2	3		1.7			1	8	
	19	L		[31] 38		0		[35] 2			[31] 1.7		0.15	1.7	
	17	L		[31] 40		Much		3			[31] 1.7		0.15	1.2	
	12	L, E, T	39			1	7/4			1.6			1, 2		
	12	L, E, T	39			1	13/4	5		1.6			1, 2	5, 10	
	20	L, E, T	39			0	0/4		>8	1.6			2, 3		
		do	39			0	17/4		8	1.6			2, 3		
		L, E, T	39			0	0/4		>8	1.6			2, 3		
		do	39			0	17/4		8	1.6			2, 3		
		L, E, T	39			0	0/4		>8	1.6			2, 3		
		do	39			0	17/4		8	1.6			2, 3		
	100	do	39			6	35/4	4		1.6			1, 2	6, 12	
	100	L, E, T	39			9	65/4	4		1.6			1, 2	do	
	99	do	39			0	1/4		>8	1.6			2, 3		
	99	L, E, T	39			1	2/4		>8	1.6			2, 3		
	7	L	35			16			3	1.6			1, 2		
	9	L	32							1.6			1, 2		
	12	L	32							1.6			1, 2		
	186	L, E	32		57										
	310	L, E	32		57										
	15		28						5	1.5			1		4
1		L	33				25/2		4	3			0.4		(1)
2.6		L	27			>10	>25/2		3	3			0.4		(1.2)
2.6		L	27							3			0.4		
	1	L	35				10/8	>8	12	3			1		(1)
		T, E						9							
		T, E				Failed			1						
	44	L, E	32			5	13/3		5	3			1		2
	25	L	40		80	27	34/2	3.7		[37] 7		[37] 10	0.3	0.5	
	85	E, R	39							do			0.3		
5.7		L, R	42		80	25	25/1		4	2.5			0.8		4
1.1		L	42			0	0/1		10	2.5			0.8		1.4
9.4		L, E, R	36		70	0	0/1		10	2.5			0.8		1.2
0.8	1.5	L	49						8.5	[43] 6			0.9		0.9
		L	49						8.5	do			0.9		0.9
	16	L	37			Some	7/6		8.5	[45] 6			0.6		0.5
	20	L	28				11/15		8.5	do			0.6		0.3
	2.7	L	30				3/3		8.5	do			[illegible]		0.5
11.3	40	E, L, R	36	[31] 47	66		10/4	>4		0.9	[31] 1.1	1.7	0.5		
1.8	20	E, L, R	36	do	66		do	>4		0.9	do	1.7	0.5		
1.9	9.5	E, L. R	36	47	66		10/4	>4		0.9	[31] 1.1	1.7	0.5		
29	146	L	[44] 19			Minor			10	[47] 2			1.25		1.2
2	10	L, E	25					4		1.25			0.5	2.4	
0.5	2	L	43		53		50/4	2		0.8		1.6	0.5	10	

Table 2.—Tabulation o

Agency	Test-site data							Thermoplastic-striping data			
		ADT [2]		Pavement							
	Project and location	Total	Per lane	Type [3]	Age [4]	Snowplow activity	Approximate mean annual snowfall [5]	Date striped	Pavement pretreatment [6]	Primer used	Name of thermoplastic [7]
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	(K)	(L)
											STATE OR TERRITORY—
		Thousands	*Thousands*		*Years*		*Inches*	*Month-year*			
New York	I-81, Jefferson Co. (FIM 61-4)	5	1.5	B	New	Heavy	60-100	10-61		Synthetic rubber.	Crys
	I-81, Jefferson Co. (FIM 61-4)	5	1.5	B	New	do	60-100	10-61		Synthetic rubber.	do
	Southern State Parkway, Suffolk Co. (SSP 62-3).	55		C	2-24	Moderate	24-36	10-63	Air	Epoxy [46], [45]	do
	Hecksher State Parkway, Suffolk Co. (HSPM 63-2).	42		C	1-3	do	24-36	10-63	do	do	Perm
	Meadowbrook State Parkway, Nassau Co. (MSP 62-3).	38		C	Old and new.	do	24-36	9-64	do	do [45]	do
		38		B	do	do	24-36	9-64	do	Synthetic rubber.	do
	Fire Island Inlet Bridge, Suffolk Co. (FIIB 64-1).	4.2		C	New	Moderate	24-36	10-64	do	Epoxy [45]	do
		4.2		B	do	do	24-36	10-64	do	Synthetic rubber.	do
	Northern State Parkway, Suffolk Co. (NSP 65-2).	60		C	Old	do	24-36	9-65	do	Epoxy [45]	do
	I-81, Jefferson Co. (FIRCM 65 134).	6		B	New	Heavy	55	10 65	brm	Synthetic rubber.	Cat
	U.S. 20, W. of Albany	12.5		C	19	do	60-100	5-60	do	do	do
	I-81, Oswego Co., (FIM 61-1)	10		C	1-3	do	60-100	7-61	do	do	Perm
	I-87, Saratoga Co., (FIM 61-2)	7		C	0.25 1	do	60-100	11-61	do	do	Crys
	I-490, Monroe Co., (FIM 61-3)	36		C	0.25-2	do	60-100	10-61	do	do	do
Ohio	Akron, Youngstown, Ravenna	High		B, C	Old and new.	do	36-60	11-64	Clean	None	Cat
Oklahoma	I-35-3(30)126, Oklahoma City		7	B	New	None	6-12	8-61	None	Epoxy	Perm
	SAP-16(9), U.S. 62 in Lawton		4	C	New	do	6-12	8-66	None	do	do
	I-440-4(26)154		5	C	do	do	6-12	4-61	do	do	do
	I-35-4(47)142, S. of Guthrie		2.8	C	do	do	6-12	10-61	do	do	do
Oregon	I-5, Minnesota Freeway, Portland		8.5	C	1	do	12	9-65	do	Pliobond	Cat
	U.S. 99E, Salem		4	B	Old	do	12	9-65	do	do	do
	U.S. 99E, Salem		4	B	New	do	12	9-65	do	Pliobond	do
	U.S. 20, Santiam Pass		1	B	Old	Heavy	100-300	9-65	do	do	do
Rhode Island	I-195, East Providence		5.2-11.5	B	New	do	36	10-59	do	None	do
			5.2-11.5	B	do	do	36	10-59	do	do	do
	I-95, Providence		13.3	B	do	do	36	10-63	do	do	Perm
	I-95, Providence		13.3	B	New	do	36	10-63	do	Pliobond	do
	I-95, Cranston, Providence		11-14.3	B	do	do	36	10 65	do	do	do
	I-95, Providence, Warwick		6.7-11.3	B		do	36	9-66			do
South Carolina	I-26-4(24) 175, Harleyville to Ridgeville.	3.8	1	B,C	New and 0.5.	None	9	9-62	Buff	Synthetic rubber.[48]	Cat
	I-385-2(28) 58, Greenville	7.5	1.9	B	.04	1 per yr.	4	9-62	do	do	do
Texas	I-45, Gulf Freeway, Houston (District 12).	87.6	14.9	C	10	None	0	4-59	brm	None	Perm
		87.6	14.9	B	10	do	0	4-59	do	do	do
		87.6	14.9	C	10	do	0	3-60	do	Unknown	Cat
		87.6	14.9	C	10	do	0	4-59	do	do	do
		87.6	14.9	B	10	do	0	4-59	do	do	do
Wisconsin	I-94, East-West Freeway, Milwaukee.	72	12	C	3-4	Moderate	36-60	8-66	None [50]	Epoxy	Perm [50]
		72	12	B	4	do	36-60	8 66	do	do	do
	I-894, Milwaukee	32	8	C	New	do	36-60	8-66	sbl	do	do
Wyoming	Casper (City Street)		2.3	B	Old	Unknown	72	9-65	None	Unknown	Unknown
	I-25, Casper		2.2	B	Old	do	72	9-65	do	do	do
District of Columbia.	Wisconsin Ave., NW		5.2	B	New	Medium	12-24	9-60	do	None	Perm
			5.2	B	do	do	12-24	9-60	do	do	do
	Georgia Ave., NW		4.2	B	do	do	12-24	9-62	do	do	Crys
			4.2	B	do	do	12-24	9 62	do	do	do
Puerto Rico	PR-23, Rio Piedras, F. D. Roosevelt Ave.		8.4	B		None	0	3-64	do	do	Perm
	Fernandez Juncas Ave	36	9	B		do	0	11-66	do	do	Unknown
	Balderioty de Castro Ave	43		B		do	0	10-64			do
	Ponce de Leon Ave	37		B		do	0	1-65			do
	State Rd. 1, Rio Piedras to Caguas.	74	18	C		do	0	12-63		Epoxy Pliobond	Perm
											TOLL
Illinois State Toll Highway Commission.	E-W Tollway, Plaza 61		3	C	Old	Heavy	12-36	?-65	brm	Epoxy	Perm
	N-W Tollway, Plaza 19		3	C	do	do	12-36	?-66	do	do	do
	do Plaza 33		3	C	do	do	12-36	?-66	do	do	do
New York State Thruway Authority.	Roadway			C			36 100	6-60			Cat
	Ramp			C			36-100	6-60			do
Port of New York Authority.	John F. Kennedy Airport		9.7	C, B		Active	24-36	10-67	None	Epoxy	Perm
Triborough Bridge and Tunnel Authority.	General experience						24-36				

See footnotes at end of table.

survey results—Continued

Amount of striping		Type of stripe [5]	Thermoplastic-striping data—Continued: Installation cost per linear foot of stripe [9] (P)			Amount of stripe lost (Q)		Average useful life (R)		Standard paint-striping data [7]: Materials and installation cost per linear feet [10] (S)			Average useful life	Ratio of long-term cost, thermoplastic: paint [15]	
(M)	(N)	(O)	4-inch width	6-inch width	8-inch width	First year	Total lost	Actual [12]	Estimated [13]	4-inch width	6-inch width	8-inch width	(T)	Determined	Estimated [16]
Continued															
Miles	*Thousands of linear feet*		*Cents*	*Cents*	*Cents*	*Percent*	*Pct./No. of yrs*	*Years*	*Years*	*Cents*	*Cents*	*Cents*	*Years*		
	150	L	26			1	50/5.5	5.5		1.7			0.5	1.4	
		E	26				15/5.5		10	1.7					
	304	L	30			1	10/4.5		10	1			0.4		
	250	L	30			1	5/4.5		10	1			0.4		
	34	L	35				13/3.5		8	1			0.4		
		L	35				1/3.5		12	1			0.4		
	49	L	35				5/3.5		10	1			0.4		
		L	35				1/3.5		12	1			0.4		
	174	L	34			1	5/2.5		10	1			0.4		
	600	L, E	32.5			0	[46] 5/2.5		10	1.7			0.6		
	3	L				100		0.5		1.7				>26	
	111	L	22.5			10–20	69/3.2	3		1.7			1	2.4	
	248	L	26			60	95/3	1		1.7			1.5	23	
	88	L	26			70–80	95/2.3	1		1.7			0.8	12	
	17	L, G	49		82		20+/2		3.5	3		13	0.5		2.3, 0.9
	90	L, E, G, T	30				6/3		7	[46] 3			0.33		0.5
	35		28			0	0/1		6	[47] 3			0.4		0.5
	9	L, E, G	30				17/4		7	do			0.4		0.5
	92	L, G	30				6/3		7	do			0.9		1
	13	L	33				1/2		5.5	1.5			0.25		1
	11	L	33				15/2		3.5	1.5			1		6
		L	33				1/2		>8	1.5			1		
	10	C	33				40/2		2.5	1.5			0.5	4	
	82	L	29	48	60		?/8	8		[47] 1.8			0.5	1	
		E	29	48	60		?/8		12	do			0.5		0.7
	118	E	31	48	60				8	do			0.5		1
	118	L	31	48	60				>8	[47] 1.8			0.5		<1
	302	E,L	31	48	60				7	do			0.5		1.3
	358	E,L	31	48	60					do					
	24	L	33							1.5			1.9		
	8	L	33							1.7			1.6		
	19	L	[48] 23				42/6	4		1.7			0.33	1	
		L	do				13/6		>8	1.7			0.33		
	18	L	[49] 29				35/5	4		1.7			0.33	1.4	
		L	do				21/6	4		1.7			0.33	1.4	
		L	do				8/6		>8	1.7			0.33		
	132	L, G	32		60	[50] 27			2	[51] 4		[51] 6	0.5		
		L, G	32		60				>5	do		do	0.5		<1
	8.4	L	32		60					do		do	0.5		
	0.2	L	63					>2		6			0.1		<0.5
	0.1	L	63						4	6			1		2.5
4.3		C,L	25				40+/7	7	4+	[31] 3			0.4	1	
4.3		T	28				25/2.5	6		[31] 5			0.25	0.4	
4.7		C,L	22				40+/5	5		[31] 3.3			0.4	0.7	
4.7		T	25					4		[31] 6			0.25	0.3	
	13.7	L	32				22/3.5		6	3.7			0.33	0.7	
		L					2/1		>10						
							15/3		8						
							2/2		>10						
		C,L					70/4								
ROADS															
	4.8	T			65		8/2	2				3	0.5	5.5	
		T			65		8/1		2			3	0.5		5.5
		T			65		8/1		2			3	0.5		5.5
		L				100		<1							
		G				100		<1							
	19	L	[52] 42						4.5	2.4			0.25		1
		L	37					3		5			0.5		1.2

Table 2.—Tabulation o

Agency	Project and location	Test-site data: ADT [2]: Total	ADT [2]: Per lane	Pavement: Type [3]	Pavement: Age [4]	Snowplow activity	Approximate mean annual snowfall [5]	Thermoplastic-striping data: Date striped	Pavement pretreatment [6]	Primer used	Name of thermoplastic [7]
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	(K)	(L)
CITIES AND											
		Thousands	*Thousands*		*Years*		*Inches*	*Month-year*			
Atlanta, Ga	General observations, Central Business District.			C, B	New, old		1-2	?-64	None	Epoxy [53]	Perm
				C, B	do		1-2	?-54	do	do	do
Baltimore, Md	do						12-24				
Detroit, Mich	(See Mich. State Highway Department, Chrysler Freeway).										
Los Angeles County, Calif.	General observations		3-12	C, B	New, old	None	0	?-61	brm [55]	Pliobond	Perm
New Orleans, La	do						0				Cat
New York, N.Y	do		13	C			24-36	?-58		Epoxy	Perm
			13	B			24-36	?-58		None	do
Portland, Oreg	N.E. 33rd Ave		7	B	New	None	12	9-65	None	Epoxy-Ply	Cat
San Francisco	General observations, city streets	5-10					0				
	General observations, downtown	15-40					0				
	General observations, downtown, heavy turning.	15-40					0				
	Van Ness Ave	35	6				0	5-62			

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[1] Under same or equivalent conditions of exposure as thermoplastic.

[2] Average daily traffic during time of stripe exposure.

[3] B=bituminous, C=concrete.

[4] At time of thermoplastic installation.

[5] Obtained from reporting agency and Weather Bureau records, or estimated for location from Climatic Maps of the United States, U.S. Department of Commerce, Revised 1966 (fig. 11).

[6] sbl=sandblast, air=air blower, abr=abraded, acid=acid etch, brm=broomed.

[7] Perm=Permaline, Cat=Catatherm, Crys=Crystallex, unkn.=unknown.

[8] L=lane line, C=center line, E=edge line, T=transverse and stop lines, R=ramp edge line, G=gore or channelizing line.

[9] Contract basis unless otherwise stated. Generally includes material, labor, equipment depreciation, profit, etc.

[10] Unless otherwise stated, installed by agency forces and include cost of materials, labor, equipment depreciation, etc.

[11] From data on cost per linear foot per year of useful life for thermoplastic and paint.

[12] Terminal point reached in test section.

[13] Estimated from still serviceable stripe.

[14] Where useful life of still serviceable thermoplastic has been estimated under column R.

[15] For the lower density section averaging 4,000 ADT per lane.

[16] Cost was actually 45¢ per linear foot but estimated to be 35¢ in larger applications.

[17] Last observation made in 1965.

[18] Contract price. Cost if done by State was estimated to be about 1.4¢ per linear foot for 4-inch width.

[19] On new portland cement concrete only.

[20] On portland cement concrete only.

[21] Salt and sand used but only light plowing.

[22] State composition specification material supplied by DeSoto Chemical.

[23] Applied by State forces.

[24] Pavement to be resurfaced.

[25] *High* traffic density.

[26] Adhesive Products Corp. material marketed as ADOPOX. Two parts ADOPOX Resin T-243 R-2 plus one part Hardener T-166 H-1.

[27] Contract prices.

California Division of Highways required that all new concrete be given a light sandblasting and estimated that the cost of this practice was about $100 a stripe-mile. The Minnesota Highway Department, which had considerable recent experience, indicated that concrete should be given some sort of light grinding to promote better adhesion of the thermoplastic.

Use of primers before thermoplastic application

Different types of primer pretreatment were used by the various agencies for both bituminous and concrete surfaces, including synthetic-rubber and epoxy treatments; some agencies used no primer at all. In many instances the agency did not know the type of primer used, or even whether any was used at all.

On bituminous surfaces, the most common practice was to apply thermoplastic striping to unprimed pavement. Some synthetic-rubber primers and even some epoxy resins were used. The New York Department of Transportation reported no difference in the performance of thermoplastic on bituminous pavements, regardless of whether synthetic-rubber or epoxy solutions were used. According to a report by the Georgia Institute of Technology, priming is not an essential prerequisite for bituminous surfaces (*5*). In summary, the omission of a primer did not seem to affect the durability of thermoplastic when it was applied over bituminous surfaces.

On concrete surfaces, the most prevalent primer used in recent years has been epoxy resin solutions. Synthetic rubber has been used to a lesser extent in recent years. In only a very few installations was no primer used at all on concrete surfaces, and early failure was reported for at least one of these. The need for adequate priming of concrete was stressed by the Arizona Highway Department and in the above-mentioned Georgia Tech report (*5*). It was reported that epoxy primers provided greater adhesion than the rubber-based primers, as was evident in the survey replies from the States of Kentucky, New York, Connecticut, and others (*5*). In a few installations in Minnesota and Puerto Rico—epoxies, when compared to rubber-based primers, did not significantly improve adhesion. Only the State of Nebraska reported that rubber-based primers provided better service than epoxy primers. On the whole, the survey results indicated that epoxy resin primers were preferable for concrete pavements, but that much more improvement is needed in the entire technology to assure proper adhesion.

Application rate of primers

The amount of primer used prior to thermoplastic striping may be a factor in good bonding, especially on concrete surfaces. The most prevalent application rate for rubber-based primers was 50 sq. ft. per gallon. In several States it was believed that adhesion could be improved if the amount of primer were increased.

For epoxy primers, the approximate application rate was about 320–420 sq. ft. per gallon, which is roughly equivalent to 800–1,200 lin. (linear) ft. per gallon for a 4-inch stripe. This amount of primer provides a wet film thickness of approximately 4–5 mils. In one State a much lower application rate of 4,000 lin. ft. per gallon, or roughly 1 mil wet thickness, gave a poorer bond on con-

survey results—Continued

Thermoplastic-striping data—Continued										Standard paint-striping data [1]					
Amount of striping		Type of stripe [8]	Installation cost per linear foot of stripe [9] (P)			Amount of stripe lost (Q)		Average useful life (R)		Materials and installation cost per linear feet [10] (S)			Average useful life	Ratio of long-term cost, thermoplastic/paint [11] (U)	
(M)	(N)	(O)	4-inch width	6-inch width	8-inch width	First year	Total lost	Actual [12]	Estimated [13]	4-inch width	6-inch width	8-inch width	(T)	Determined	Estimated [11]
COUNTIES															
Miles	*Thousands of linear feet*		*Cents*	*Cents*	*Cents*	*Percent*	*Pct./No. of yrs.*	*Years*	*Years*	*Cents*	*Cents*	*Cents*	*Years*		
25		L, C	[54] 17						4	3.1			0.5		0.7
		T		[54] 25				1.8			?		0.25		
	40				[56] 70			4				[56] 7.2	1	2.5	
			25						6	2			0.5		1
			28							[57] 10				0.5	
	1,000	L	28							do					
	4.6	L	28			<1			10	0.9			1		3
		L, T								1.3		[56] 13	2		
		L, T								1.3		[56] 13	1		
		L, T								1.3		[56] 13	0.5		
		L, T	60		[56] 60				>5	1.3		[56] 13	1		

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[28] Plus additional installation cost.
[29] Deterioration began in 2 months with blistering and adhesion loss.
[30] 3-inch width.
[31] 5-inch width.
[32] Based on State's estimate of thermoplastic: paint cost ratio of 10:1 and performance ratio of 16:1.
[33] 4½-inch width.
[34] Replaced by warranty in June 1966.
[35] Failed because of movement of bituminous overlay on portland cement concrete.
[36] Occasional brooming but not definite.
[37] Only plowed once in winter of 1966–67.
[38] Preceded by prime of synthetic resin compound.
[39] Paved shoulder.
[40] Permaseal III H (Cook Paint & Varnish Co.) applied at rate of 4,000 linear feet per gallon.
[41] Removal of obvious dirt only.
[42] 6 to 8 gallons per mile of actual stripe.
[43] Contract price. Average cost by State forces is 3.5¢.
[44] State advises that this is a very low bid but real. Probably done at or below cost to get striping in for demonstration purposes.
[45] Maximum thickness of 6 mils.
[46] 50% loss on bituminous section without synthetic rubber primer. Only 1% loss on edge line.
[47] 5¢ by contract.
[48] 50 sq. ft. per gallon.
[49] Excludes State labor for traffic control.
[50] Thermoplastic placed over existing paint.
[51] Includes application of black paint in skip zone.
[52] Unconditional guarantee for 3 years. Believed that this guarantee contributed to higher cost.
[53] Permaseal 1 and 2 used in equal proportions on portland cement concrete surface.
[54] Done by city forces with leased equipment.
[55] Hand broom. Grind to remove old paint. Sandblast to remove curing agent on fresh portland cement concrete.
[56] 12-inches wide.
[57] Paint cost is 2¢, remainder of cost is 8¢.

crete than the higher application rate. However, when the primer was applied to bituminous surfaces, a high application rate of 1,000 lin. ft. per gallon caused the thermoplastic to *slide* over the primer resulting in poor adhesion to the pavement. Therefore, the lower rate of 4,000 lin. ft. per gallon was preferred for asphalt surfaces.

One thermoplastic producer held the opinion that the proper application rate of epoxy primer to concrete had to be different for different concrete; that is, 500 lin. ft. per gallon for new, more absorptive concrete, and about 1,000 lin. ft. per gallon for older, less absorptive surfaces. From the survey results it appeared that the optimum application rate of epoxy primer to concrete is far from resolved, and depends on the age, porosity, and texture of the pavement as well as on the active solids content of the epoxy solution used. To provide good bonding to the thermoplastic and yet not be so thick as to limit the escape of solvents and, thereby, interfere with the epoxy-catalyst reaction or contribute to the phenomenon of blistering, an optimum film thickness of primer apparently must remain on the pavement surface after all absorptive effects have taken place.

Time interval between primer and thermoplastic application

On concrete surfaces especially, the time interval between the application of epoxy binder and the application of thermoplastic seemed to be a factor in the adhesion and blistering of the marking. This interval differed considerably in the few replies that included such information. In New York State, for example, the practice was to permit the primer to dry about 15 minutes before the hot thermoplastic was applied, which allowed sufficient time for the solvent in the primer to volatilize. Shorter time intervals had a tendency to enhance the blistering of the thermoplastic so often noted in New York and other States when the striping was applied to concrete. A developing preference of contractors is the application of both the epoxy primer and thermoplastic from the same vehicle, and the use of an infrared heater to speed drying of the primer. Thus the time interval between the application of primer and the application of thermoplastic is reduced to substantially less than a minute. On the basis of subjective observations by several agencies, this practice may contribute to excessive blistering and poor adhesion. One State stipulated a maximum time interval of 30 minutes. A few States noted that blistering of the thermoplastic seldom occurred on bituminous pavements, regardless of the primer used or the extent of the subsequent drying period.

Reflectance and color properties

Information on the relative reflectance and visibility of thermoplastic and paint varied considerably. Daylight reflectance of thermoplastic decreased noticeably with age and, although still satisfactory, the thermoplastic was not as bright as fresh paint. On the other hand, several agencies noted that the night visibility of thermoplastics was somewhat better than paint, especially under wet conditions. However, no definite pattern was evident from the reports received.

Effect of pavement surface and underlying traffic paint

According to the survey, the durability of thermoplastic striping was much better on bituminous surfaces than on concrete surfaces. One exception was noted for an installation with a bituminous overlay on portland cement concrete. The shifting of the overlay by traffic caused premature cracking and failure of the thermoplastic striping.

Several agencies noted that thermoplastic performance was much better on older concrete than on new concrete. Some agencies recommended against the use of thermoplastics on new concrete, whereas a few resort to sandblasting the concrete surface to improve bonding. Evidently, both the surface laitance layer and some curing compounds can seriously interfere with good bond to new concrete.

Several agencies noted that poor adhesion resulted when old traffic paint was not removed prior to the application of thermoplastic striping. From the limited evidence available, good practice would dictate the prior removal of built-up layers of old paint.

Special problems with edge lines

Both the New York and Kentucky highway departments observed that continuous edge lines of thermoplastic tend to impound rain water and thereby perpetrate skidding hazards. They recommend cutting transverse channels at intervals in order to permit drainage of entrapped water.

Effect of snowplows on thermoplastics

Reported snowplow damage to thermoplastics was widespread in northern States having an appreciable amount of snowfall. Several agencies reported that the material is not economically feasible in mountainous areas. A few agencies reported that snowplow damage could be reduced by feathering the leading edge of the skip line; this part of the stripe was most affected by plows. Less damage was noted where the plows were fitted with shoes and raised slightly above the pavement surface.

Terminal point of thermoplastic striping

A few of the survey replies made available the agency's criteria for assessing the terminal point in the useful life of thermoplastic striping. The majority of these replies indicated that the agencies consider the terminal point to be reached when only about 40–60 percent of the material is still intact on the pavement. It would seem that when 50 percent or more of the stripe is lost, the terminal point of the stripe has been reached or exceeded—an assumption that conforms to the findings in another survey(2).

Development of Criteria for Use of Thermoplastics

The information obtained in the survey seems to warrant the development of more sharply defined criteria than the currently available and diffuse criteria shown in table 3. Suitable criteria are needed to show when thermoplastic markings instead of conventional paint striping might be used to economic advantage. The criteria must be based on the original cost and life expectancy of each type of striping and also on any other economic factors inherent in the use and maintenance, such as the traffic delays and safety considerations in highly congested areas caused by frequent maintenance striping with conventional paint. Each of these contributing parameters is analyzed separately in the following paragraphs and are subsequently integrated to provide new, sharply defined criteria. To simplify the development of this information, lane and center lines of 4-inch width, as they apply to divided highways of the Interstate type, are considered primarily. As will be shown later, the integrated criteria also will be applicable to other roadways.

Cost and life expectancy of conventional paint striping

As discussed earlier, calculations from the survey data showed that the average cost of a 4-inch-wide longitudinal stripe of conventional paint is 2.2 cents per lin. ft. of actual stripe. This cost includes all the obvious and inherent costs of striping—materials, labor, other expendable supplies, equipment depreciation, etc.—and is based on installations and maintenance striping performed by the road agency itself. It is generally applicable to open highway striping rather than to the striping of congested city streets.

The life expectancy of conventional longitudinal paint stripes was determined to be directly related to the amount of traffic exposure, as is evident in figures 2 and 3, in which the reported paint life is plotted against the average daily traffic (ADT) per lane. The data reported for bituminous pavements are shown in figure 2, and that for portland cement concrete pavements are shown in figure 3. Numerical entries at some of the data points represent mean annual snowfall, in inches, and are shown only where snowfall was significant and considerable snowplow activity expected. There was no significant correlation between the effect of snowplow operations measured by annual inches of snowfall, and paint life. In figures 2 and 3, the

Table 3.—Policy of agencies with significant experience in use of hot-applied thermoplastics

Agency	No policy stated	Thermoplastics not authorized for standard use	Criteria for use of thermoplastic in preference to conventional paint
STATE HIGHWAY DEPARTMENTS			
Alabama	X		
Arizona	X		
Arkansas			Where ADT exceeds 6,000.
California			Where ADT per lane exceeds 7,500 (urban) cr 5,000 (rural); not acceptable in heavy snow areas.
Connecticut			Roads with high traffic density.
Florida			Urban intersections with high traffic density.
Illinois			On high-volume freeways.
Indiana			On new bituminous surfaces with more than 6,000 ADT and with estimated minimum surface life of 4 years, or where excessive paint wear is experienced. Not used on portland cement concrete.
Iowa		X	
Kansas		X	
Kentucky	X		
Maine		X	
Maryland	X		
Michigan	X		
Minnesota			Economically justified on bituminous pavement.
Mississippi			Economically justified on high density roads.
New Hampshire			For stop lines and parking areas.
New Jersey		X	
New Mexico		X	
New York	X		
Ohio	X		
Oklahoma	X		
Oregon			Either: (a) 4-lane highways with ADT of more than 20,000 and 2-lane highways with ADT of more than 12,000, requiring painting 3 times yearly. Sandblast to remove old paint. Not in mountainous area. (b) Restricted or hazardous to paint area.
Rhode Island			High speed expressways.
South Carolina		X	
Texas		X	
Wisconsin			Bituminous surfaces.
Wyoming			Under consideration.
District of Columbia			Crosswalks.
Puerto Rico			High-traffic-density bituminous roads.
TOLL ROAD AUTHORITIES			
Illinois Toll Highway Commission.	X		
New York Thruway Authority	X		
Port of New York Authority			All transverse lines and longitudinal lines where economically justified.
Triborough B. & T. Authority			Material is justified even at higher cost.
CITIES AND COUNTIES			
Atlanta, Ga			For crosswalks.
Baltimore, Md			Lane lines in high-traffic-density areas.
Los Angeles County, Calif			New or recently resurfaced roads.
New Orleans, La			High-traffic-density areas.
New York, N.Y			Do.
Portland, Oreg	X		
San Francisco, Calif	X		

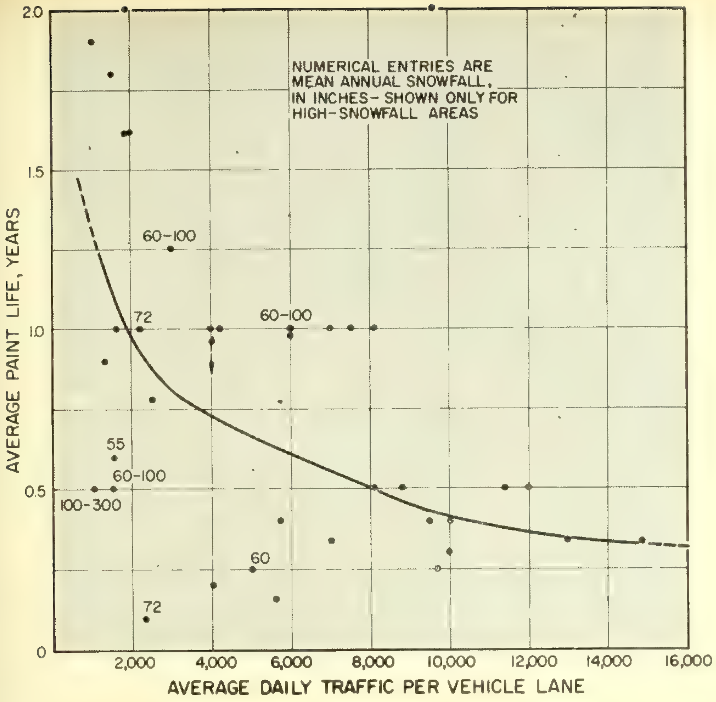

Figure 2.—Useful life of paint striping as affected by traffic density—bituminous pavement.

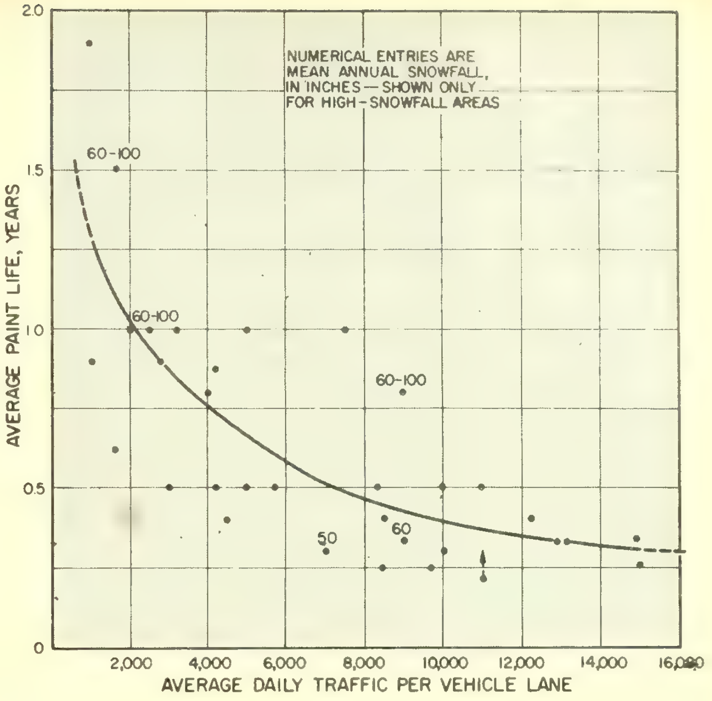

Figure 3.—Useful life of paint striping as affected by traffic-density—concrete pavement.

directional arrows pointed upward and those pointed downwards respectively represent data points reported to have the indicated minimum and maximum life expectancy. A line that best represented the average of all results plotted was drawn on each of the figures. There is very little difference in the locations of these lines on the two figures. Consequently, an average of the two lines is show in figure 4 to represent both concrete and bituminous pavements.

As previously mentioned, figures 2, 3, and 4 represent the situation for center and lane lines only. The bulk of the information received in the survey dealt with these longitudinal lines. Data obtained on other line types was insufficient to develop adequate relationships. Moreover, in the few installations for which such data was supplied, it appeared that edge lines lasted about one and one-half times as long, and transverse stripes about one-half as long as center or lane lines under similar road exposure conditions. This difference was to be expected, considering the difference in actual traffic exposure of such lines on a given highway or city street.

From the data in figure 4 and from the calculated average cost of conventional striping, 2.2 cents per lin. ft. of 4-inch-width striping, it was possible to calculate the average cost of a 1-foot length of a 4-inch wide paint stripe for a full year of useful service. The calculation was done for various traffic density levels, and the results are shown in the first three lines of table 4. As is evident in entry A, the annual cost of maintaining a paint stripe varies considerably with traffic density and can be very sizable in highly congested areas.

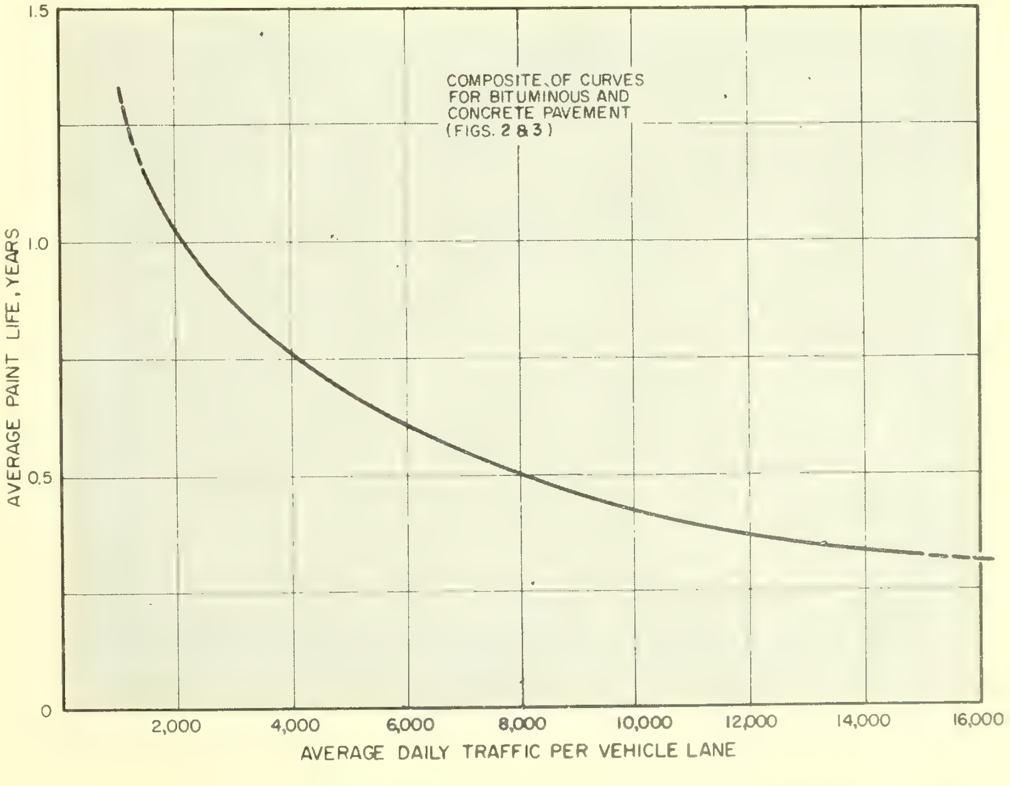

Figure 4.—Average useful life of paint striping as affected by traffic density—both bituminous and concrete pavement.

Effective costs of traffic delay during conventional maintenance striping

A significant factor in favor of long-lasting hot thermoplastic striping is that the frequency of maintenance striping is considerably reduced. Thus, the thermoplastic yields potential economic benefits in terms of reduced traffic delay caused by striping operations. As part of this study, an attempt was made to evaluate this factor and to determine the

Table 4.—Annual cost of conventional paint striping [1]

[Costs given separately and collectively for basic installation, traffic delay, and potential accidents]

	Average daily traffic (ADT) per lane—No. of vehicles															
	1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	11,000	12,000	13,000	14,000	15,000	16,000
Useful paint life [2] years	1.32	1.02	0.87	0.75	0.67	0.60	0.55	0.50	0.45	0.42	0.38	0.36	0.35	0.33	0.32	0.31
Annual striping frequency [3]	.76	.98	1.15	1.33	1.47	1.67	1.82	2.00	2.22	2.38	2.63	2.78	2.86	3.03	3.12	3.22
Annual basic paint striping cost, per linear ft. per year (A) [4] cents	1.67	2.16	2.53	2.93	3.28	3.67	4.00	4.40	4.88	5.24	5.79	6.12	6.29	6.67	6.86	7.08
Annual costs of paint striping on 4-lane divided highway:																
Cost of traffic delay, per linear ft. per year (B) [5] cents	.02	.05	.11	.16	.23	.31	.39	.49	.61	.73	.89	1.02	1.14	1.30	1.43	1.58
Cost of potential traffic accidents, per linear ft. per year (C) [6] cents	.00	.01	.01	.02	.03	.04	.05	.06	.08	.10	.12	.13	.15	.17	.19	.21
Total annual cost, per linear ft. per year (A)+(B)+(C) cents	1.69	2.22	2.65	3.11	3.54	4.02	4.44	4.95	5.57	6.07	6.80	7.27	7.58	8.14	8.48	8.87
Annual costs of paint striping on 6-lane divided highway:																
Cost of traffic delay, per linear ft. per year (D) [5] cents	.03	.07	.16	.25	.34	.46	.59	.74	.92	1.10	1.33	1.54	1.71	1.96	2.16	2.39
Cost of potential accidents, per linear ft. per year (E) [6] cents	.00	.01	.02	.03	.04	.06	.08	.10	.12	.14	.17	.20	.22	.25	.28	.31
Total annual cost, per linear ft. per year (A)+(D)+(E) cents	1.70	2.24	2.71	3.21	3.66	4.19	4.67	5.24	5.92	6.48	7.29	7.86	8.22	8.88	9.30	9.76

[1] Applicable to longitudinal striping of 4-inch wide center and lane lines, excluding edge lines, on open highways as typified by Interstate roads.

[2] Interpolated from figure 4.

[3] Calculated from data on *useful paint life.*

[4] Basic materials, labor, and installation costs calculated from: annual striping frequency × 2.2¢ (average cost per linear foot of 4-inch stripe per installation, as explained in text).

[5] Calculated from: 7.65 × 10^{-6} × total ADT × annual striping frequency (equation 1 in text).

[6] Calculated from: Total ADT × 10^{-6} × annual striping frequency (equation 2 in text).

degree to which it contributes to the overall cost of conventional striping. Some of the reasoning presented in an unpublished report[3] was used for this purpose.

It was assumed that for each mile of conventional maintenance striping on an Interstate highway, about 1 hour is required for striping, drying, and the removal of the traffic cones used to protect the fresh paint. It was also estimated that, during this time, all passing vehicles on a one-directional roadway would experience a speed reduction of approximately 20 m.p.h. Assuming a speed decrease from 55 m.p.h. to 35 m.p.h., a delay of 0.6 minutes would be imposed on each passing vehicle for each mile of striping. If this delay time is calculated in terms of linear foot of actual stripe, then the delay time becomes:

$$\frac{0.6}{5280}=.000114 \text{ min. per vehicle per linear foot of stripe.}$$

Most conventional striping is done during off-peak daylight hours. Under such conditions, one-directional traffic on urban sections of an Interstate highway has been shown to be 2.6 percent per hour of the total ADT (*9*). Thus, the *total delay, in hours*, for all vehicles affected by one linear foot of conventional on-going maintenance striping becomes:

$$\frac{.000114\times .026\times \text{ADT}}{60}=4.94\times 10^{-8}\times \text{ADT (in hours)}$$

Based on $1.55, the total hourly time cost of all the occupants in a single vehicle (*10*), the total cost of such a single delay in *cents per linear foot of striping* becomes:

$$4.94\times 10^{-8}\times \text{ADT}\times 155$$
$$=7.65\times 10^{-6}\times \text{ADT (in cents per lin. ft. of striping).}$$

The annual delay cost, given by the following expression, would depend on the frequency of restriping:

$$7.65\times 10^{-6}\times \text{ADT}\times \text{annual striping frequency (in ¢/lin. ft./year)} \qquad (1)$$

Using the above expression, equation (1), the additional annual costs of stripe maintenance attributable to traffic delays were calculated for both 4-lane and 6-lane divided highways. The results are shown in table 4 as entries B and D.

Effective costs of increased accident potential during conventional striping

Traffic-safety-benefits are an often cited intangible advantage of the more durable hot-melt thermoplastic striping. Such benefits could accrue by virtue of reduced striping frequency, thereby decreasing the accident potential that might otherwise exist because of frequent maintenance striping and its potentially hazardous effect on traffic.

An effort was made to derive some quantitative economic measure of this potential hazard and apply it to the overall cost of conventional striping. Consultation with several prominent traffic accident researchers, as well as a formal search of the available literature through the Highway Research Board's Information Service, failed to disclose any definitive literature relating to the increased accident potential that exists during a road maintenance or striping operation. However, several reports were available that did permit an empirical derivation to be made of the increased accident potential. This derivation and other considerations in the development of an economic measure of accident potential are discussed in the following paragraphs.

It has been established that the speed variance of individual vehicles from the mean speed of traffic contributes to accident involvement on Interstate highways, as well as on other main rural roads (*11, 12*). During a conventional paint striping operation, it is customary to require a speed reduction on the highway, which is usually accomplished by warning and speed-reduction signs placed in advance of the actual work. Regardless of the advance distance or number of signs; the starting point of deceleration, deceleration rate, and actual extent of vehicle speed reduction depends largely on the individual drivers. Although no measurements are known to exist for such a situation, general experience indicates that a greater-than-normal variation in speed among vehicles does prevail under such circumstances. A brief empirical analysis was made to determine what effect this increased speed variance might have on the accident involvement potential. Data from a published report (*11*) were used for this purpose.

Results of an earlier study (*9*) indicate that the standard deviation of speed on between-interchange mainline units of Interstate highways is 7 m.p.h. In the vicinity of a striping operation, it was assumed that the standard deviation of speed would be somewhat higher, perhaps from 10 to 15 m.p.h. Using the speed data in table 1 of Interim Report II (*11*), the involvement rate relating to accidents was computed for these situations.

From these computations it was found that the theoretical increase in accident involvement rate over that for normal traffic operation with a standard deviation of 7 m.p.h. is 3.5 if the standard deviation is assumed to be 10 m.p.h., and 22.8 if the standard deviation is assumed to be 15 m.p.h. Thus, in the vicinity of a pavement-marking operation, the involvement rate may increase from 3.5 to 22.8 times the involvement rate during normal traffic operations. From this range, a value of 10 times the normal involvement rate was arbitrarily selected as an approximation of the potential increase in involvement

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[3] *Study of Warranting Conditions for Use of Thermoplastic Lane Markers*, Bureau of Traffic, Ohio Department of Highways, 1967. (Unpublished.)

As most striping operations occur during off-peak daylight hours, the normal accident involvement rate was calculated for this period. From table 1, Interim Report II (*11*), the average or normal involvement rate during daylight off-peak hours was calculated to be 54.4 per 100 million vehicle miles (MVM)[4] on an Interstate highway. This rate is applicable only to accidents involving two or more vehicles traveling in the same direction and does not include single vehicle run-off-the-road type accidents. However, from a previous Interstate study, it was established that about one-third of all accidents involved two or more cars traveling in the same direction, and that the remainder involved only a single vehicle (*13*). Thus, if 54.4 is the involvement rate for accidents with two or more vehicles, single vehicle accidents excluded, then 54.4 (or less) is the additional involvement rate attributable to single car accidents. This additional rate gives a total *normal* accident involvement rate of no more than 108.8, which should account for all one-directional traffic during off-peak daylight hours on Interstate roads. From previous considerations, the theoretical involvement rate attributed to a striping operation should be about 10 times this value—10 × 108.8 or 1,088. The net difference in involvement rate between normal operations and traffic striping operations accordingly is 1,088−108.8, or roughly 979—say a round figure of 1,000. Thus, an involvement rate of 1,000 can be used as an expression of the additional potential hazards owing to conventional paint striping on an Interstate highway. From data in an Illinois report (*14*), the average cost of a single involvement was calculated to be about $200. Hence, the dollar cost per 100 MVM for an accident involvement rate of 1,000 is:

$$1{,}000\ (\text{per }100\ \text{MVM}) \times \$200$$

If it is assumed that the hazardous area is 1 mile of directional roadway just preceding, alongside, and following the striping operation, and that this length of roadway will be a hindrance to traffic for 1 hour, as previously discussed under traffic delays, the traffic density as well as the number of vehicle-miles in this hazardous area are given by the following single expression:

$$.026 \times \text{ADT}\ \text{(during off-peak daylight hours)}$$

The potential accident cost, in dollars per mile of stripe during a single striping operation is obtained by combining the two preceding expressions, and is given by X in the equation:

$$\frac{X}{.026 \times \text{ADT}} = \frac{1{,}000 \times \$200}{100\ \text{MVM}}$$

Where,

ADT = average daily traffic
MVM = million vehicle miles

therefore:

$$X = \frac{1{,}000 \times \$200 \times .026 \times \text{ADT}}{100\ \text{MVM}}$$

Potential accident costs, expressed in cents per linear foot of conventional striping during an entire year, then become:

$$\text{Accident cost (¢/lin. ft./yr.)} = \frac{1{,}000 \times 20{,}000¢ \times .026 \times \text{ADT} \times \text{yearly striping frequency}}{100\ \text{MVM} \times 5{,}280\ \text{lin. ft.}}$$

or:

$$= \frac{10^3 \times 2 \times 10^4 \times 2.6 \times 10^{-2} \times \text{ADT} \times \text{yearly striping frequency}}{10^2 \times 10^6 \times 5.28 \times 10^3}$$

or approximately expressed as:

$$\text{Accident cost (¢/lin. ft./yr.)} = \text{ADT} \times 10^{-6} \times \text{yearly striping frequency} \qquad (2)$$

Using equation (2), the potential accident cost for conventional striping was calculated for various ADT's and is shown in table 4, entries C and E. From a comparison of these values with those in entries B and D, it is readily apparent that the additional cost of standard paint striping attributable to potential accidents is negligible, compared to the other economic factors, despite the fact that the empirically derived value for the increase in potential involvement rate over normal operations—10 times 108.8—is a rather liberal allowance, according to the subjective estimates of several accident researchers who were consulted.

Summation costs for all contributing economic factors were calculated and are shown as entries A+B+C and A+D+E of table 4. The cost analysis presented in the table will be used in a subsequent section to develop information on the comparative cost effectiveness of hot-extruded thermoplastic striping.

One additional factor, advantageous to the use of longer-lasting thermoplastics, is the reduced exposure of maintenance forces to traffic hazards during restriping operations. It was not possible to obtain quantitative data or develop empirical considerations that could be used to translate the advantage into

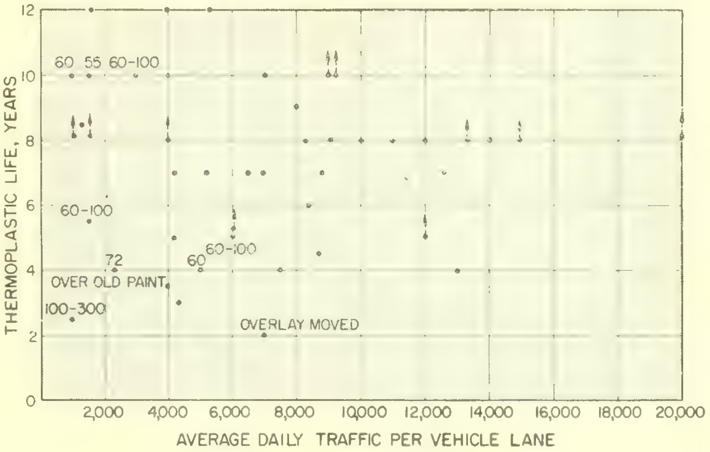

Figure 5.—Relation between thermoplastic durability and traffic density—bituminous pavement.

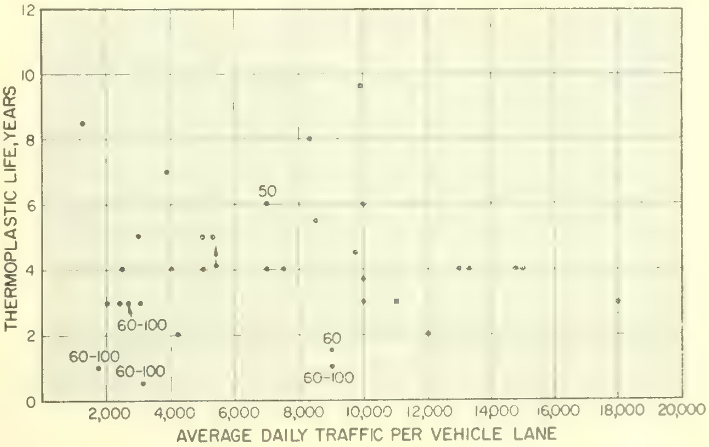

Figure 6.—Relation between thermoplastic durability and traffic density—concrete pavement.

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[4] Involvement rate is the number of involvements per 100 million vehicle-miles, and implies a vehicle involved in an accident. Thus, one accident involving two vehicles is counted as two involvements.

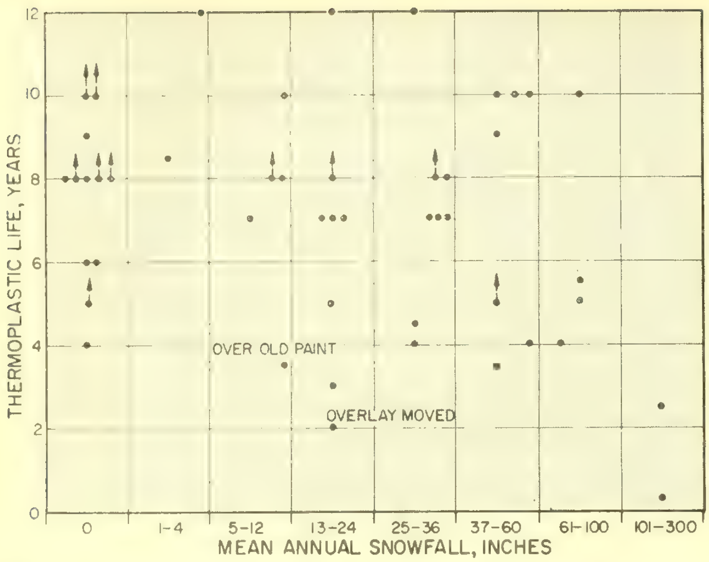

Figure 7.—Relation between thermoplastic durability and annual snowfall—bituminous pavement.

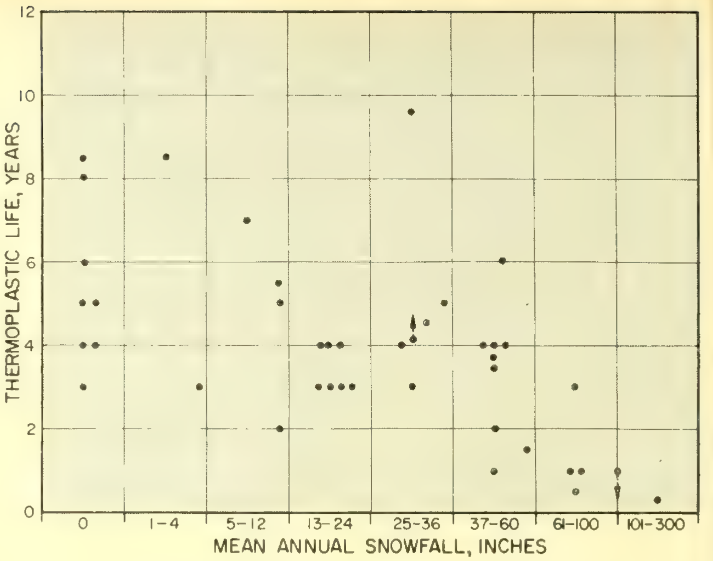

Figure 8.—Relation between thermoplastic durability and annual snowfall—concrete pavement.

potential economic value with quantitative dimensions. Therefore, the economic considerations must remain a subjective factor at this time. It should be remembered however, that although thermoplastic striping is required less frequently, this advantage is somewhat offset by the fact that it is a much slower operation than conventional striping, and exposes the striping crew to traffic hazards for a period two to three times as long as that required for paint striping during any single striping operation.

Cost and life expectancy of hot-melt thermoplastic striping

As stated earlier the average cost of thermoplastic striping reported in this survey was 32.7 cents per linear foot of 4-inch stripe. In general, this cost is for installations performed on a contract basis, the most prominent method of installation.

No correlation could be found between theromoplastic life expectancy and traffic density, as was evident in conventional striping. This lack of correlation is shown in figures 5 and 6 for bituminous and concrete surfaces, respectively. Theoretically, some relation would be expected, but so many other variables affected performance that such a relation was obscured. Interfering variables that possibly affected durability of the thermoplastic were snowplow operations, pavement pretreatment, primer type and application rate, and pavement age. Of these, the only single parameter that showed some independent correlation with thermoplastic durability was the intensity of snowplow operations as measured by mean annual snowfall. Plots of annual snowfall data against the reported useful life of thermoplastics are shown separately for bituminous and concrete pavements in figures 7 and 8. These relations are more clearly evident in figure 9 in which individual data for each snowfall grouping was averaged, and the average plotted. Numerical entries within each box (bituminous pavement) or triangle (concrete pavement) indicate the number of individual data points that were averaged to obtain the plotted result, and the curves were carefully weighted to reflect these values. It is apparent from figure 9 that a correlation does exist between thermoplastic durability and mean annual snowfall for each type of pavement. It is also evident that thermoplastics are much more durable on bituminous pavements than on concrete. On both types of pavements, durability was decreased in high-snowfall areas. These results agree completely with the individual observations reported by many of the highway agencies.

The curves in figure 9 apply mainly to center and lane lines. Where additional data were available, it seemed to indicate that edge lines lasted about one and one-half times as long as the indicated durabilities and transverse lines about one-half as long.

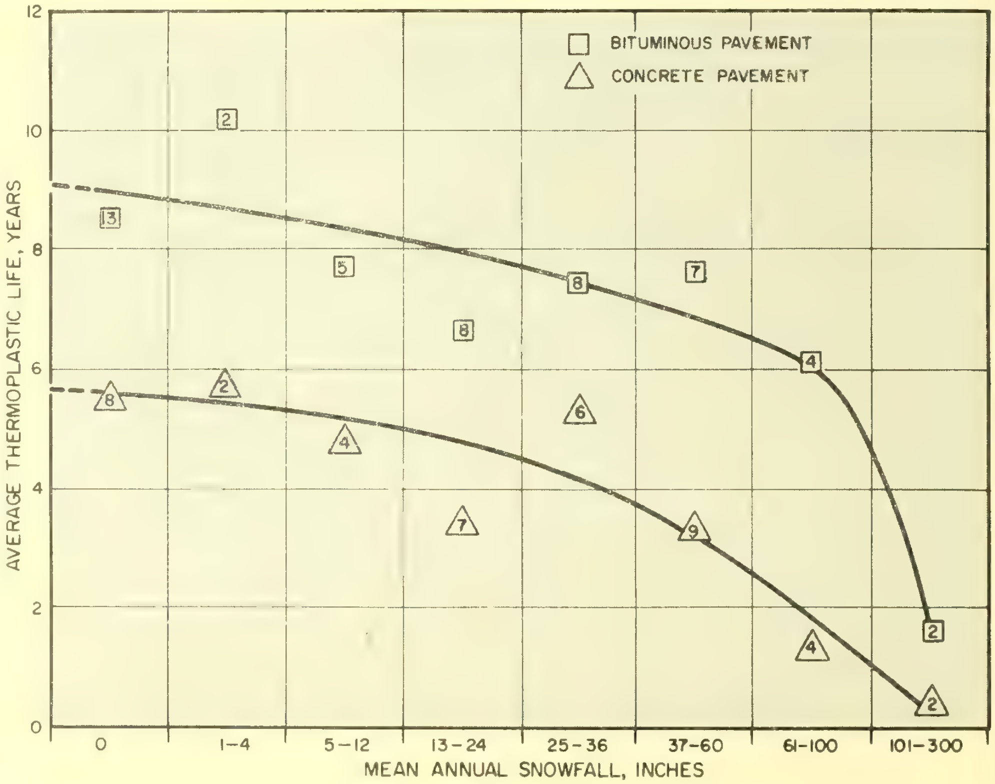

Figure 9.—Relation between average thermoplastic life and annual snowfall.

Guide for selecting the most economical material

On the basis of the data already presented, it was possible to develop criteria for selecting the most economical striping material—either paint or hot-extruded thermoplastic.

Data on the annual unit cost of paint striping, as it was affected by traffic density is provided by table 4. Cost data are given for the installation alone as well as for additional economic factors caused by frequent maintenance striping—traffic delay and traffic safety. The objective was to determine those conditions under which thermoplastic striping had costs comparable to conventional striping, as well as lower and higher costs. The average life of thermoplastic striping was interpolated from figure 9 for each pavement type at the midpoint of each incremental snowfall grouping. From these interpolated values and the average cost of the thermoplastic installation, 32.7 cents per linear foot, the unit costs per year of service were calculated for each pavement type and degree of snowfall. These costs were matched against interpolated costs for paint for the various traffic densities (table 4). It was then possible to establish equal cost *matches* for paint and thermoplastics, and the exact conditions under which they were operative—ADT, snowfall, and pavement type.

As a result of this calculation, a new chart, shown in figure 10, was constructed to show the conditions under which the long-term cost of painting and thermoplastic striping were equivalent. Points along curves C and F of the figure represent conditions conducive to equal costs of the two materials when the actual installation costs are considered alone and the economic effects of traffic delays and potential accident hazards are disregarded. Curves A and D are lines of equivalent costs for 6-lane divided highways, and curves B and E are for 4-lane divided highway. Any combination of snowfall and traffic density conditions that falls to the left of an appropriately selected demarcation curve would be an indication that paint is more economical than thermoplastics. The opposite is true for a combination of conditions that are to the right of the selected curve. The lower portion of the three curves for bituminous pavement—A, B, and C—have dashed vertical segments which are *cut-off points* that were determined as described in the following paragraph.

A number of road agencies reported that after the thermoplastic stripe had been used for some period, the bituminous surface required overall maintenance, necessitating the thermoplastic striping, which was still serviceable, to be covered over by a bituminous topping. The value of any long-lived stripe is governed by the maintenance-free life of the bituminous pavement. The following comments of the various agencies were selected as illustrative of the expected maintenance-free life of bituminous pavements.

Alabama—"Bituminous pavement required resurfacing after 8 years."

Arizona—"Use thermoplastics with caution on bituminous pavements because of limited maintenance-free life expectancy of such surface."

Kentucky—"Estimate maintenance-free life expectancy of bituminous pavements not to exceed 8 to 10 years."

Oklahoma—"4 to 5-year-old bituminous surface was resurfaced resulting in the obliteration of the thermoplastic marking."

Los Angeles County—"With road maintenance and utility work, the useful life of the pavement surface is not much more

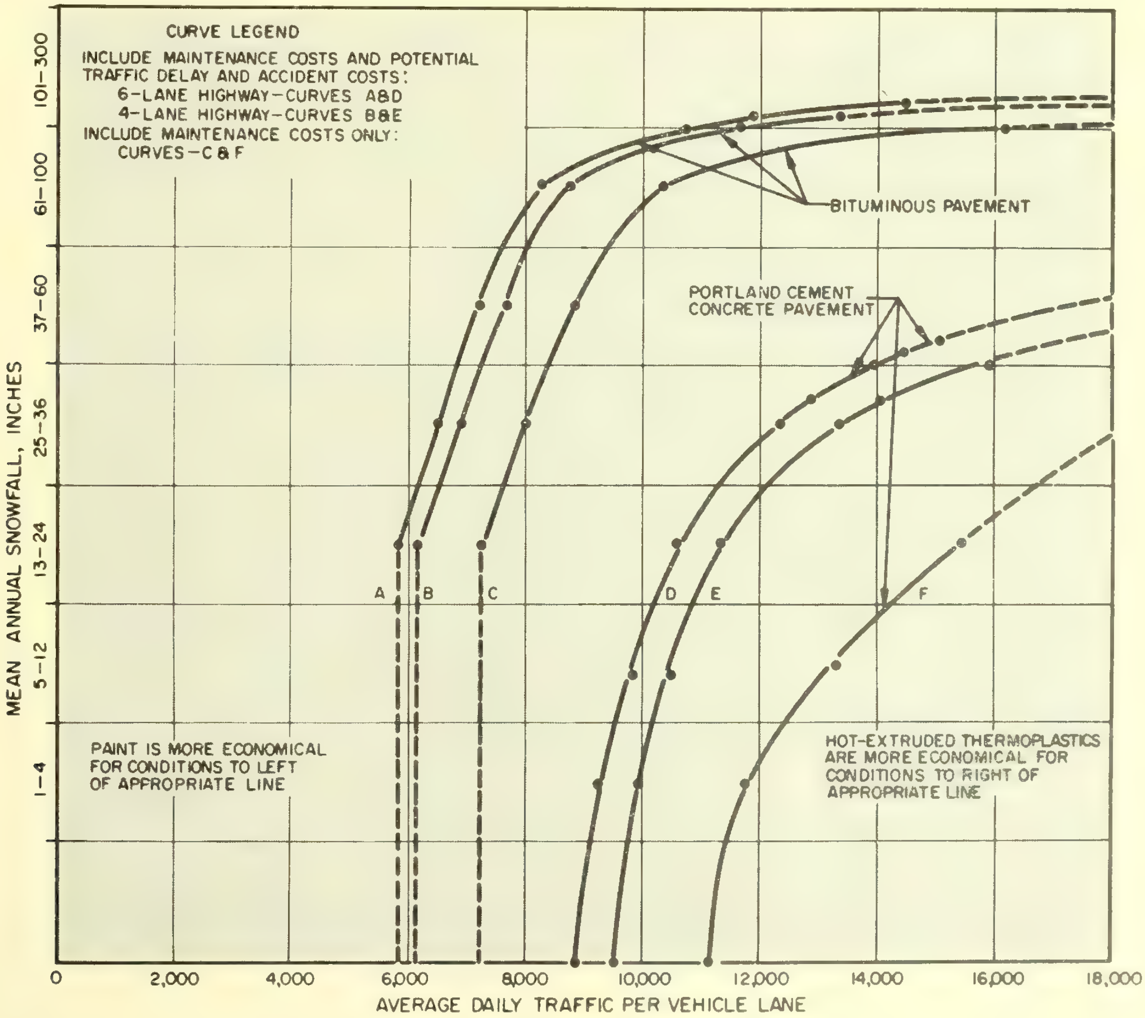

Figure 10.—Guide for selecting the most economical striping material, paint or thermoplastic.

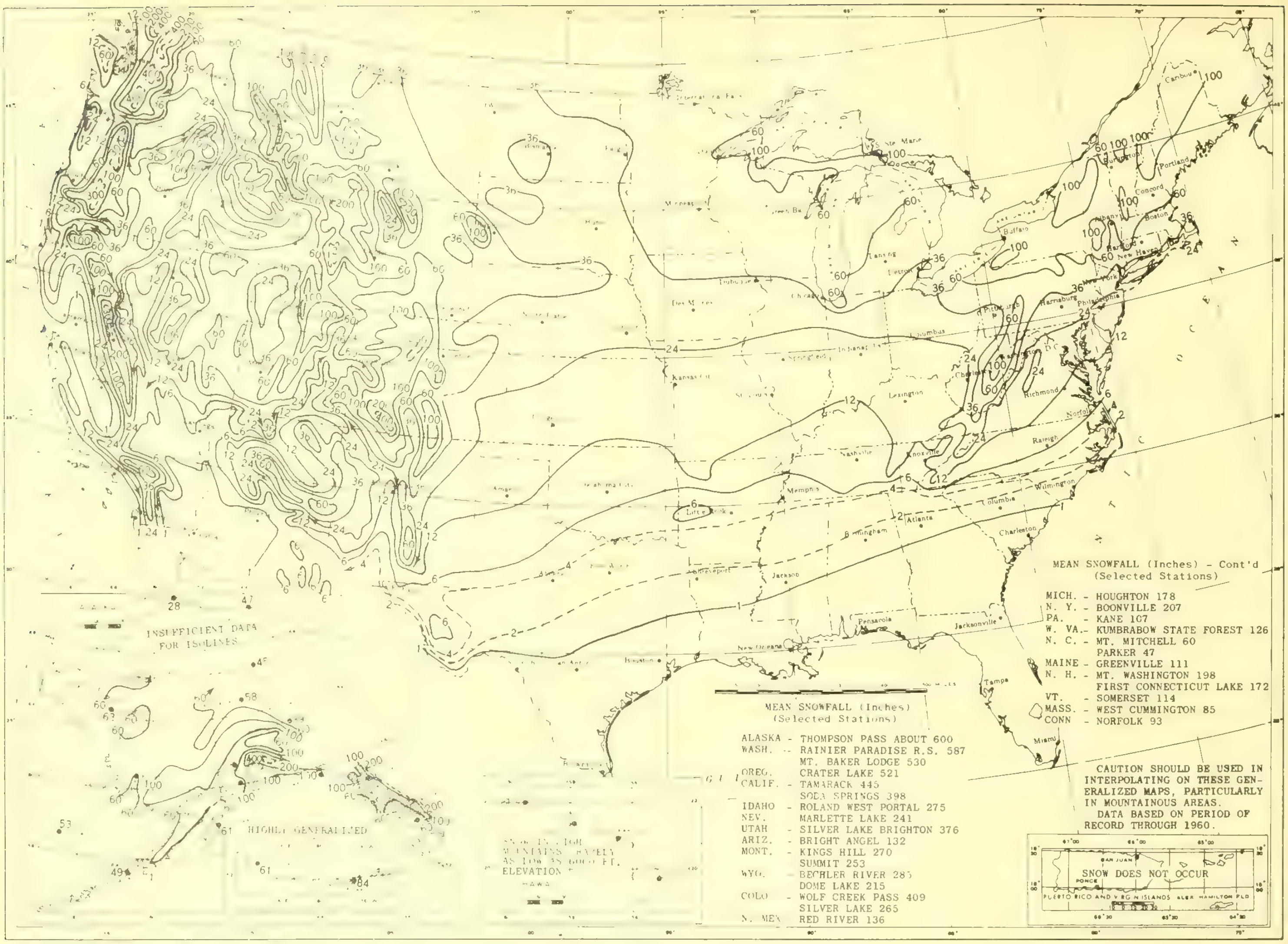
INSUFFICIENT DATA FOR ISOLINES
HIGHLY GENERALIZED
MEAN SNOWFALL (Inches)
(Selected Stations)
ALASKA - THOMPSON PASS ABOUT 600
WASH. -- RAINIER PARADISE R.S. 587
MT. BAKER LODGE 530
OREG. CRATER LAKE 521
CALIF. - TAMARACK 445
SODA SPRINGS 398
IDAHO - ROLAND WEST PORTAL 275
NEV. MARLETTE LAKE 241
UTAH - SILVER LAKE BRIGHTON 376
ARIZ. - BRIGHT ANGEL 132
MONT. - KINGS HILL 270
SUMMIT 253
WYO. - BECHLER RIVER 285
DOME LAKE 215
COLO - WOLF CREEK PASS 409
SILVER LAKE 265
N. MEX RED RIVER 136
MEAN SNOWFALL (Inches) - Cont'd
(Selected Stations)
MICH. - HOUGHTON 178
N. Y. - BOONVILLE 207
PA. - KANE 107
W. VA.- KUMBRABOW STATE FOREST 126
N. C. - MT. MITCHELL 60
PARKER 47
MAINE - GREENVILLE 111
N. H. - MT. WASHINGTON 198
FIRST CONNECTICUT LAKE 172
VT. - SOMERSET 114
MASS. - WEST CUMMINGTON 85
CONN - NORFOLK 93
CAUTION SHOULD BE USED IN INTERPOLATING ON THESE GENERALIZED MAPS, PARTICULARLY IN MOUNTAINOUS AREAS.
DATA BASED ON PERIOD OF RECORD THROUGH 1960.
SAN JUAN
PONCE
SNOW DOES NOT OCCUR
PUERTO RICO AND VIRGIN ISLANDS

Figure 11.—Mean annual snowfall, in inches, in the United States (prepared by the Department of Commerce, Revised 1966).

> than 4 years, and thermoplastics are therefore limited to this."

Public Roads' specialists were also consulted on the durability of bituminous pavement surfaces. According to the information obtained from all sources, the best approximation of the average maintenance-free life of a bituminous surface was about 8 years. Therefore, this period was established as the nominal maximum useful life of thermoplastic striping on bituminous pavement and was the basis used to calculate the lowest possible annual unit cost applicable to thermoplastic striping, thus establishing the limiting value shown by the dashed vertical segments of figure 10.

Applicability and use of guide chart

Figure 10 can be used as a guide to determine whether conventional paint or hot-melt thermoplastic is the most economical striping for a given location. This chart is based mainly on data obtained on 4-inch-wide longitudinal lane and center lines on open highways and is essentially applicable to installations in which such lines are used. However, these criteria should be applicable to other stripe widths, provided that the ratio of installation costs for paint and thermoplastic remain the same as found in this survey. The criteria should also be applicable to edge markings, provided that the ratio of useful paint life to thermoplastic life is similar to that for lane or center striping.

To use the criteria for a given location, the following information is required to judge the relative economics of the two striping materials:

- Estimated or actual ADT per vehicle lane.
- Type of pavement surface—concrete or bituminous, and number of vehicle lanes.
- Mean annual snowfall in inches.

Mean annual snowfall can be obtained from local Weather Bureau officials, or from the snowfall contour lines shown on the Climatic Maps of the U.S., published by the U.S. Department of Commerce, Environmental Science Services Administration, Environmental Data Service. A reproduction of such a map is shown in figure 11.

Using the above information and the guide chart, figure 10, plot the appropriate values for mean annual snowfall (in inches) against the ADT per vehicle lane. Select a demarcation line to denote both pavement type and whether installation costs only or the additional economic factors of traffic delay and safety considerations are to be considered. Curves C and F are used to compare only direct tangible costs—installation and maintenance. Curves A, B, D, and E are used to compare overall costs that include traffic delay and potential accident costs. Curves A and B apply to bituminous surfaces—curve A for 6-lane, curve B for 4-lane roads. Curves D and E apply to concrete surfaces—curve D for 6-lane, curve E for 4-lane roads. Data points falling to the right of a selected demarcation line indicate that thermoplastics are more economical than paint for the conditions to be encountered. Points falling to the left indicate that paint is more economical than thermoplastics. When thermoplastics are selected for use under these criteria, their application should be considered carefully. The installation of thermoplastic on new concrete without precleaning or sandblasting the concrete surface is a risky operation. For older bituminous pavements, the remaining years of maintenance-free service expected of the pavement surface should be determined and this period balanced against expected stripe life.

The guide chart is applicable when the ratio of total installed cost of thermoplastic to paint is approximately 15:1, as determined in the reported survey. A cost ratio that differs substantially from 15:1 should be taken into account by appropriate modification of the described criteria. The guide chart does not consider the hazard to maintenance crews during restriping operations. This is a subjective factor to be weighted in favor of thermoplastics.

Comparison of Developed Criteria With Other Available Guides

The proposed criteria for the selection of the most economical striping material, developed in the study reported here and illustrated in figure 10, were compared with the limited guides suggested by several agencies in their response to the survey. (See table 3.) The agencies' guides, essentially, are qualitative guides that limit the use of hot-melt thermoplastics to high density roads and/or bituminous surfaces. For such installations, the proposed quantitative criteria are compatible with these qualitative restrictions. The guides offered by the Arkansas and Indiana highway departments are much more liberal than those developed here. They permit the use of thermoplastics, at least on asphalt pavements, when the *total* ADT is at least 6,000. A minimum limit of 6,000 ADT *per vehicle lane* is suggested by the proposed criteria. The criteria of California and Oregon are more in line with the proposed criteria. In these States, the minimum ADT per vehicle lane conducive for the best economic use of thermoplastics is approximately 5,000 to 6,000.

An important feature of the proposed set of guides is that it provides for continuous quantitative criteria which cover a wide range of snowfall and traffic conditions and type of pavement surface.

Typical Specifications and Warranties for Thermoplastic Materials

Incidental to the primary information developed from the survey, some data were obtained on typical specifications and warranties used for hot-melt thermoplastics. This information, summarized in tables 5 through 9, is incomplete and is not representative of all practices, but it does provide an adequate sampling of what is generally available and in use.

A summary of typical specifications for synthetic rubber primers used in conjunction with hot-extruded thermoplastics is given in table 5. These primers had been used largely for portland cement concrete surfaces but currently are being gradually replaced by epoxy primers. According to the data in the table, the critical components are either a mixture of neoprene and butadiene-styrene rubber or of nitrile rubber (Buna N) and phenolic resin. These materials usually are supplied and used in solution form containing about 10 to 20 percent solids by weight.

Information on typical specifications for two-part epoxy primers, which have begun to replace the synthetic rubber primers, is listed in table 6. Generally, each of the two components is used in solution form containing about 50 percent reactive solids.

A tabulation of typical specifications for hot-extruded thermoplastic striping material is presented in table 7. The material, referred to as Crystallex, is apparently designed to meet British Standards, as is evident from the tabulation. The material, which is specified by the California Highway Department, is a specialized formulation that is somewhat different from the other materials listed. The

Table 5.—Typical specifications for synthetic rubber primer for thermoplastics

Agency or company	Goodyear Tire & Rubber Co.	Calif. State Highway Dept.	N.Y. State Highway Dept.	Conn. State Highway Dept.
Specification	*Technical Book Facts* PB-6.	63-F-40	White thermoplastic, Item 401.	Special Provisions, Federal-Aid Projects I-91-2(35)31 and I-91-3(53)36.
Specification date	Received May 1968.	1963	Apr. 18, 1968	July 15, 1963.
Name of material	Pliobond 20[1]	Primer for traffic paints and thermoplastic traffic paints.[2]	Primer for bituminous concrete pavement.	
Composition (solids basis):				
Neoprene ... percent			Required (or SBR).	Required (or SBR).
Butadiene-styrene (SBR) ... do			Required (or Neoprene).	Required (or Neoprene).
Buna N rubber ... do		60–70		
Phenolic resin ... do		30–40		
Synthetic rubber ... do	Required [1]			
Synthetic resin ... do	Required [1]			
Conform to standard infrared spectrum.		Required		
Solids ... percent	20	9–11	10 [3]	10 [3].
Weight per gallon ... pounds	7.2			
Solvents (based on total solvent):			Volatile, organic	Volatile, organic.
MEK ... percent	Required	50		
MIBK ... do		20		
DIBK ... do		5		
Toluene ... do		20		
Xylene ... do		5		
Dry time, tack-free ... minutes			Less than 5 (6 mil film at 70° F. and 60% R.H.[4]).	

[1] In qualitative infrared analysis by Public Roads laboratory, it was shown that this material is a Buna N-phenolic resin mixture similar to the standard spectrum in California State specification 63-F-40.

[2] Intended for use on new portland cement concrete surfaces prior to application of traffic paint and on portland cement concrete and asphaltic concrete surfaces prior to application of thermoplastic traffic marking material.

[3] Minimum.

[4] R.H. = Relative humidity.

Table 6.—Typical specifications for 2-part epoxy primer for thermoplastics

Agency or company	Adhesive Products Corp.	N.Y. State Highway Dept.	Cataphote Corp.	Perma-Line Co.
Specification		White thermoplastic, Item 401.		Concrete Sealer-Binder DX-1037.
Specification date	Sept. 7, 1967	Apr. 18, 1968	Nov. 22, 1967	Received May 17, 1968.
Name of material	Adopox	(1)	(1) (2)	Perma-Seal III.[3]
Epoxy component (A)	T-243R-2 Adopox			Bisphenol.
Reactive solids ... percent	50		50 [4]	52.
Weight per gallon ... pounds	8.095	8.095 [4]	8.10 [4]	8.35±1.
Epoxide equivalent (solids basis)	185–200	185–200	185–200.	
Solvent(s)	Aliphatic			Aromatic.
Viscosity, No. 2 Zahn, 77° F seconds				14±1.
Appearance and color				Clear, straw yellow.
Catalyst component (B)	T-166H-1 Adopox Hardener.			
Type				Aromatic amine.
Reactive solids ... percent	50		50 [4]	26.
Weight per gallon ... pounds	7.405	7.405 [4]	7.40 [4].	7.47±0.1.
Solvent(s)	Aliphatic			Aromatic
Viscosity, No. 2 Zahn, 77° F seconds				14±1.
Appearance				Clear, red-brown.
Mixture of (A) and (B):				
Mix ratio (A:B), by volume	2:1	2:1	2:1	1:1.
Pot life (closed container at 72° F).	3–4 days	24 hours [4]	24 hours [4]	2–4 hours.
Dry time, to tacky ... minutes	15 (5–7 mils wet at 40–110° F).			
Color of dried film	Clear			
Reactive solids content, pct. by weight		50 (A+B, 1:1 vol.).[4]		30–41 (A+B, 1:1 vol.).

[1] Primer for portland cement concrete.

[2] For application prior to thermoplastic, apply between 40–110° F. at 5–7 mils wet. Apply marking material after it is tacky, or approximately 30 minutes under normal conditions, 70° F. and 40% R.H.

[3] Apply at 50–100° F. Sandblast portland cement concrete where required.

[4] Minimum.

Table 7.—Typical specifications for hot-applied thermoplastic material

Agency or company	Perma-Line Co.	Cataphote Corp.	Constructex Overseas Ltd.	British Standard (B.S.)	California State Highway Department	Florida State Highway Department	Connecticut State Highway Department	New York State Department of Transportation	
Specification	Perma-Line	Catatherm	Crystallex [1]	B.S. 3262, Part 1, 1960.	White thermoplastic IR 353.	Special provisions to 1966 standard specifications, Item 611.	Special provisions for highway I-91.	White thermoplastic.	White thermoplastic.
Specification date	Revised Mar. 7, 1968 (long form).	Received Aug. 29, 1967.	Received May 17, 1963.		January 1964.	Nov. 21, 1966.	May 1964	Apr. 18, 1968.	Mar. 25, 1966.
LABORATORY PROPERTIES									
Composition:									
Pigment, for white thermoplastic ... percent by weight	6-10 (of pigment).[2]	10 minimum (of pigment).[2]		6-10 minimum.[2]	13.0 [3]		10 minimum (of pigment).[2]	10 minimum.[2]	10-15.[2]
Extender pigment ... do				10-14 [5]	41.6 [4]				
Aggregate ... do	(3)	(3)		38-42				38-49	32-48.
Beads (premixed) ... do	15-50	25-50		18-22	20.8	20 [6]	20-30	20-25	20-25.
Organic binder ... do	15-35	15-35		18-22				17-22	22-28.
Synthetic resins	(7)	(7)		Required					
Modified alkyd						Required			Required.
Hydrogenated ester gum ... do					9.8				
Pliolite AC ... do					9.0				
Hydroabietyl alcohol ... do					5.8				
Rosin				Required					
Mineral oil				do					
Monohydric primary alcohol								Required [8]	
Modified maleic resin								Required	
Color:									
Yellow (FTMS 141, Method 4252), match standard color chip	Required	Required [9]				Required			
White (ASTM E-97) Daylight reflectance (Rd) minimum ... percent	70	70			75 [10]	70	70		
Redness greenness, a	-5 to +5	-5 to +5				-5 to +5	-5 to +5		
Yellowness blueness, b	-10 to +10	-10 to +10				-10 to +10	-10 to +10.		
Color retention, no perceptible change									
Yellowness index, maximum (FTMS 141, Method 6131).					0.12 [13]				
After heating to plastic state	4 hours [11]	4 hours [12]			(16)				
After ultraviolet exposure ... hours	(14)	100 [15]				100 [15]	(15)	100 [15]	100.[15]
Specific gravity (25° C./25° C.)	1.0-2.5	1.9-2.5	2			1.9-2.15		1.9-2.2	1.9-2.0.
No deterioration when heated to plastic state ... do	4	4 [17]			4 [17]	4 [15]	(18)		
Toxic fumes, when heated to plastic state	None	None				None			
Volatile material							None		
Temperature-viscosity characteristics, the same after reheating ... No. of reheatings	4	4			(17)	4			
Water absorption (ASTM D 570) maximum ... percent	0.5	0.5				0.5			
Softening point (ASTM E 28, Ring and Ball) minimum ... degrees C	90	90	(19)	40-50		90		90	90.
Impact resistance (ASTM D 256) at 77° F., 1- x 1- x 3-inch cast bars, minimum ... inch-pounds	10								
Bond strength (ASTM C 321) between 3- x 7-inch area of portland cement concrete ... minimum p.s.i.	150	150 [20]			200 [16] [22] [23]				
Indentation resistance (ASTM D 1706), Shore Durometer Type A-2, reading after heating 4 hours at 400° F., cooled and held for 15 seconds at:									
115° F ... reading	60 [21]	65			47-57 [16] [25]				
77° F ... do	95 [21]	95 [24]							
40° F ... do		95							
Cracking, low temperature stress resistance (coating on portland cement concrete surface) no cracking, flaking or adhesion failure.					(26)	(27)			
Flowability, residue in can ... percent					13-17 [28]				
ROAD PROPERTIES AFTER APPLICATION									
Deterioration by deicing chemicals, pavement constituents or oil drippings.	None	None				None			
Deformation or discoloration under normal traffic	None [29]	None [30]				None [31]	None [32]		
Drying time, no impression or imprint by traffic:									
At 50° F ... minutes	2 [33]	2 [33]			2 [34]				
At 70° F ... do						15 [35]		15 [36]	15 [36]
At 90° F ... do	15 [33]	15 [33]			10 [34]				
Free from tack	Yes	Yes				Yes			
Chipping or cracking	None	None				None			
Slippery when wet	No	No				No	No		
APPLICATION REQUIREMENTS									
Temperature, pavement, minimum ... degrees F				50 [36]		(37)	40 [38]	40 [39]	40 [39]
Pavement condition				Dry, no old paint.			No paint	Dry, oil-free.	Dry, oil-free.
Pavement precleaning, remove dirt, oil, grease.	Yes [40]	Yes [40]					Yes [40]		
Primer:	Required	Required				Required [40]			
For portland cement concrete pavement	do			Required [41]		do	Required [42]	Required [42]	Required [42]
For [illegible] pavement	Required [43]	Required [43]				do [44]		do [45]	Do.[45]
Primer application rate (wet film) ... sq. yd. per gal				70-100.					
Synthetic rubber ... sq. ft. per gal							50.		50.
Epoxy ... do		(46)						429	230-320.

Table 7.—Typical specifications for hot-applied thermoplastic material—Continued

Agency or company	Perma-Line Co.	Cataphote Corp.	Constructex Overseas Ltd.	British Standard (B.S.)	California State Highway Department	Florida State Highway Department	Connecticut State Highway Department	New York State Department of Transportation	
APPLICATION REQUIREMENTS—Continued									
Time between primer and thermoplastic application:									
With synthetic rubber				After solvent evaporates					
With epoxy ... minutes		30[47]					30–120	15[48]	15[48]
Thermoplastic application temperature ... degrees F	375–475	380–420	280–350[49]	250–280		380–420		360–420	360–420.
Thermoplastic application speed ... m.p.h.	2[6]						4 in.—17–171 ft/min	15,000–20,000 lin. ft./8 hour	15,000–20,000 lin. ft./8 hour.
Thickness of thermoplastic in place:									
Center, minimum ... inch	1/8	1/8							
Edge, minimum ... do	3/32	3/32							
Overall ... do	3/32–5/32	3/32–3/16		1/8		3/32–3/16	3/32–1/8	1/8–3/16	1/8–3/16.
Drop-on beads, application rate, per 100 sq.ft. of line ... pounds	3, 20[50]					6.2	3	5	5.
Visibility				60[51]					
CERTIFICATION OF TEST COMPLIANCE									
Certification of compliance by contractor						Required	Required		

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[1] Crystallex specifications similar to B.S. 3262, Part 1.
[2] TiO_2.
[3] $CaCO_3$ or lithopone.
[4] $CaCO_3$.
[5] $CaCO_3$, white, 5,000 p.s.i. minimum.
[6] Minimum.
[7] Mixture (one must be a solid at room temperature).
[8] High boiling point.
[9] Federal yellow.
[10] After 4 hours at 450° F.
[11] At 395° F. and 4 reheatings.
[12] At 375° F. and 4 reheatings.
[13] Repeated reheatings.
[14] Prolonged exposure to sun.
[15] ASTM D 620.
[16] And 4 reheatings.
[17] At 450° F. and reheatings at 450° F.
[18] 4 reheatings.
[19] 45 for standard grade; 60 for tropical grade.
[20] ⅛-inch thick with binder.
[21] 1/16–1/8-inch thick.
[22] Portland cement concrete blocks sandblasted and primed with 63-F-40 primer.
[23] 1/16–1/8-inch thick.
[24] Minimum (2-pound weight).
[25] 2-kilogram weight.
[26] ⅛-inch layer on 1 square foot sandblasted and primed portland cement concrete, conditioned, to 77° F., 24 hours at 15° F., remove and examine within 5 minutes.
[27] 32-square-inch specimen, 1 hour in cold H_2O, 24 hours at −20° C., condition and test at ambient temperature.
[28] 20% residue after 4 hours at 450° F.
[29] Provided air and road temperature is between −20° and 120° F.
[30] Provided air and road temperature is between 0° and 120° F.
[31] Provided road temperature does not exceed 140° F.
[32] Provided air and road temperature is between −30° and 120° F.
[33] Maximum (maximum 70% RH).
[34] Maximum.
[35] Maximum (⅛–3/32-inch thick).
[36] Minimum pavement temperature.
[37] Articles 6.1, 6.2, 6.3, 6.10.
[38] Minimum (relative humidity 80% minimum).
[39] Where necessary.
[40] Epoxy as recommended by thermoplastic manufacturer.
[41] 1 part asphalt, 1 part dichloromethane.
[42] Epoxy.
[43] If less than 8% asphalt.
[44] If less than 7% asphalt.
[45] Synthetic rubber.
[46] 5–7 mils.
[47] Or until tacky.
[48] Approximately, till tacky.
[49] 280 for standard grade; 350° for tropical grade.
[50] High intensity beads.
[51] Minimum (average luminance).

remainder of the specifications are somewhat similar, and appear to be patterned after the two most prominently used hot-melt thermoplastics in this country—Permaline and Catatherm. The specifications are presented under separate subheadings to show clearly the required laboratory properties, field properties, and application requirements for these materials.

Typical specifications currently in use for both premixed and drop-on type beads used in conjunction with thermoplastic striping are listed in table 8. Except for the British Standard, these specifications are similar, but do show some minor differences.

Some typical warranties furnished by the contractor or required of him in connection with the durability of thermoplastic striping are tabulated in table 9.

Conclusions and Recommendations

The principal findings and recommendations developed from the study reported here are summarized in the following statements:

- Hot-extruded thermoplastic striping is much more durable on bituminous pavements than on portland cement concrete pavements.
- Thermoplastic striping generally is more durable on older concrete pavements than on new concrete.
- Snowplow activity, as measured indirectly by mean annual snowfall data, greatly affects thermoplastic adhesion to the pavement, especially on portland cement concrete. The service life of thermoplastic striping is related more to snowplow activity than to traffic density. By contrast, the durability of conventional paint striping is related to the volume of traffic.
- A limiting factor in the economic value of thermoplastic striping on bituminous pavements is the maintenance-free life of the bituminous surface; this was estimated to be an average of about 8 years.
- Unremoved layers of old traffic paint may adversely affect the adhesion of thermoplastic striping to the pavement.
- A guide (fig. 10) was developed to assist in the selection of the more economical of the two materials, conventional paint or hot-melt thermoplastics, for specific conditions: pavement surface, traffic density, and expected snowfall in the area concerned.
- Under conditions of little or no snowplow activity, thermoplastics can provide economic benefits over paint striping on bituminous pavements when the traffic density is approximately 6,000 vehicles per lane or greater, or on concrete pavements when the density exceeds 9,000 vehicles per lane. Under moderate snow conditions thermoplastics can be justified at higher traffic density levels. Little economic justification exists for the use of thermoplastics under severe snow conditions requiring considerable snowplow activity.
- Although many such installations have performed well, the greatest deterrent to the wide use of thermoplastic striping is its sporadic, and sometimes unexplained, failure on concrete surfaces. Research to improve this situation should be sharply emphasized. Suggested for such investigation are the following parameters and their contributions to thermoplastic performance on concrete: surface cleaning and preparation, improved primer formulation, rate of primer application and its relation to the age and nature of the concrete surface, time interval between primer and thermoplastic application, and feathering of thermoplastic leading edges and sides to reduce snowplow destruction.

Table 8.—Typical specifications for glass beads for thermoplastic striping

Agency or company		Perma-Line Co.	Cataphote Corp.	British Standard	California State Highway Department	Florida State Highway Department	New York State Highway Department
Specification			Data Sheet	BS 3262, Part 1.	Thermoplastic Formula, IR 353.	Spec. Prov. to 1966 Std. Specs., Item 611.	White thermoplastic. Item 401.
Specification date		Revised Mar. 7, 1968.	Received Aug. 29, 1967.	1960	January 1964.	Nov. 21, 1966.	Apr. 18, 1968.
PRE-MIXED TYPE							
Index of refraction, liquid immersion at 25° C., minimum.		1.50	1.65		1.50	1.50	1.65.
Roundness (ASTM D-1155), True spheres, minimum.	pct	70	90	80	75	70	90.
Air inclusions	do		None		None		
Milky, black, amber, or colored particles	do		1[1]				
Gradation (ASTM D-1214), passing Sieve No.:							
10	do			100[2]			
16	do						100.
20	do						98–100.
30	do						75–90.
36	do			10[1,2]			
40	do	80–100.	80–100		95[2]	80–100.	
50	do						15–40.
70	do				10[1]	0–10	
80	do	0–10	0–10				
100	do						0–10.
200	do						0–5.
Chemical resistance (water, acid).		Required					
Crushing resistance (ASTM D-1213).	pounds	40[4]					
DROP-ON TYPE							
Index of refraction, liquid immersion at 25° C., minimum.		1.50, 1.90[5]	1.65				1.65.
Roundness (ASTM D-1155), True spheres.	pct	70[5]	90.				90.[3]
Milky, black, amber, or colored particles.	do		1[1]				
Gradation (ASTM D-1214), passing Sieve No.:							
16	do						100.
20	do	80–100[5]	90–100				98–100.
30	do						75–90.
35	do		0–10				
40	do						
50	do	90–100					15–40.
70	do	0–10					
80	do	0–10					
100	do						0–10.
200	do						0–5.
Chemical resistance (water, acid).		Required					
Moisture resistance, cotton bag-funnel test							Required.
Crushing resistance (ASTM D-1213).	pounds	40[4]					

[1] Maximum.
[2] British Standard (BS) sieve.
[3] Minimum.
[4] Average minimum.
[5] When high intensity stripe is specified.

Table 9.—Typical warranties for hot-applied thermoplastics

Agency or company		Perma-Line Co.	Cataphote Corp.	New York City
Specification		Perma-Line.	Catatherm.	Plastic Marking.
Specification date		Mar. 7, 1968	Received Aug. 29, 1967.	1968.
Crosswalks and stop lines, percent guaranteed for:				
1 year	percent	90[1]	90[1]	100.[4,5]
1½ years	do			
2 years	do	75[1,2]	75[1,3]	
[illegible] years	do			
3 years	do		50[1]	
Lane and center lines, percent guaranteed for:				
1 year	do	90[2]	90[2,3]	100.[5,7]
[illegible]	do			100.[5,8]
[illegible]	do	80[2]	80[3]	100.[5,9]
[illegible]	do			100.[5,10]
[illegible]	do	60[2]	60[3]	100.[5,11]

[1] Except at the first [illegible] ft. [illegible] each intersection.
[2] [illegible] 5,000 ADT [illegible]
[3] [illegible] 30,000 ADT [illegible]
[4] School crossings.
[5] When more than [illegible] occur [illegible] within guarantee period, [illegible] Contractor not liable for damage caused by snowplow blades.
[6] Percent of a unit defined as 2,000 linear ft. of line of specific width.
[7] More than 40,000 ADT.
[8] 30,000–40,000 ADT.
[9] 20,000–30,000 ADT.
[10] 10,000–20,000 ADT.
[11] Less than 10,000 ADT.

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SURVEY ON PERFORMANCE OF HOT-EXTRUDED THERMOPLASTIC STRIPING MATERIALS FOR HIGHWAYS—DETAILED INFORMATION

Data Requested in Survey

- Location of controlled experiment.
- Length of installation (linear feet of thermoplastic or other basis).
- Date initiated.
- Average daily traffic per vehicle lane.
- Pavement surface (asphalt or concrete, new or older surface).
- Pretreatment of pavement surface (other than primer).
- Primer applied, if any (synthetic rubber, epoxy, etc.).
- Brand name of thermoplastic used.
- Relative snowplow activity.
- Unit cost of thermoplastic stripe (contract cost or other basis)—cost per linear foot of actual stripe (indicate stripe width—4, 6, 8, etc., inches).
- Comparative unit cost of applying a similar width of conventional traffic paint stripe (either contract cost or cost including materials, labor, and equipment depreciation).
- Average actual useful life of conventional paint stripe in area under consideration.
- Average useful life of thermoplastic stripe in this same area (indicate whether estimated on basis of performance of still useful stripes or whether terminal point was reached by this material).
- Estimates of percentage of thermoplastic line lost by adhesion failure.
- Cost of replacing thermoplastic (indicate basis of cost).
- Comments and specific conclusions on relative durability and long-term economy of these competitive materials in this specific area and for these circumstances. Include statement of any special conditions that may have affected behavior on this project.
- Present policy, practice and criteria employed relative to the use of hot-extruded thermoplastics in regular maintenance striping operations.

The following note was included in the survey questionnaire: If general knowledge is available, based on experiences with several installations that may not be included as parts of controlled experimental projects, please provide a general summary of such knowledge.

Comments by Agencies Surveyed

States, District of Columbia, and Puerto Rico

Alabama.—High traffic-volume portion of heavy duty bituminous concrete resurfaced after 8 years of service, covering over thermoplastic and not permitting full life of thermoplastic striping. Limitations of conventional paint are diminished crew safety because of more frequent maintenance and poor visibility during last half of paint's useful life. After 8 years, 30 percent of original ⅛-inch thick thermoplastic remains on section with 4,000 average daily traffic per lane. No adhesion failures noted.

Arizona.—In 5 years of service, reflective properties of thermoplastic have deteriorated considerably and stripe does not remain bright throughout its life. Material should be used with caution on bituminous pavement owing to flexibility and shorter life expectancy of bituminous surface. Caution in using on portland cement concrete with less than 1 year of service because of moisture, curing compounds, and laitance. Earlier tests showed thermoplastic performed poorly on portland cement concrete without primer.

Arkansas.—Thermoplastic lane lines applied over 3-week-old paint stripe on portland cement concrete. Air bubbles formed initially. Night visibility superior to paint. No bond failures in 2 years, but about 8 percent chipped at edges. Better service on bituminous pavement than on portland cement concrete pavement.

California.—New portland cement concrete should be sandblasted (a cost of approximately $100 per mile). Thermoplastic lasts indefinitely on bituminous pavement owing to fusion. Most failures on portland cement concrete occur from chipping because of poor adhesion. Although thermoplastic was better than paint in many areas, it is being replaced by raised markers in snow-free areas.

Connecticut.—Average life of paint reported as 2 months but actually repaint only 2 to 3 times a year. Ten percent loss of thermoplastic in last year on portland cement concrete replaced by contractor. More wear on horizontal curves. Section of portland cement concrete primed with synthetic rubber is much poorer than that primed with epoxy. Placement velocity of thermoplastic is 5 m.p.h., on long continuous lengths. Blistering still a problem over portland cement concrete pavement. Thermoplastic not subject to color fading as with paint. Unexplained isolated cases of failure still occur. Practice is to apply epoxy primer and thermoplastic from same moving truck using heater pass over the epoxy. May change this to allow more time for primer to dry.

Florida.—Gainesville (University Avenue) Catatherm much softer than Perma-Line. Yellow Catatherm bleaches to lighter shade. Good to excellent night reflectivity, but poorer and dirty in day. Dirt accumulates in depressions but appears better after a rain.

Interstate 95 (Dade County)—In 2 months thermoplastic began to deteriorate over portland cement concrete but paint still satisfactory. Thermoplastic blistering and breaking away. Better durability on older bituminous pavement. *Northern* grade of thermoplastic in Miami failed because of shifting and softness. Thermoplastic on portland cement concrete showed air pockets and blisters.

Illinois.—Terminal life of thermoplastic is considered reached when only 50 percent remains.

Indiana.—Currently not using thermoplastic on portland cement concrete because of adhesion failures. Have had satisfactory experience on portland cement concrete in isolated cases.

Iowa.—Ninety days after installation, thermoplastic appearance was dull. First application on portland cement concrete failed within 9 months through adhesion loss. Replacement under warranty performing adequately. High cost cannot be justified, considering economics of standard paint over portland cement concrete.

Kansas.—Movement of bituminous overlay on portland cement concrete caused early thermoplastic failures.

Kentucky.—Suggest that feasibility of use of thermoplastic be estimated on basis of anticipated renewals of traffic paint during a reasonable period—not exceeding tenure of particular pavement surface and certainly not more than 8 to 10 years. Thermoplastic loss of more than 1 percent per year, or less than 90 percent terminal retention in line footage is intolerable. Thermoplastic performed better where greater application rate of Pliobond primer was used. Thermoplastic lost adhesion more quickly on new portland cement concrete than on older portland cement concrete. Epoxy-primed section on portland cement concrete more durable than Pliobond-primed section on portland cement concrete. Better performance of thermoplastic on bituminous pavement than on portland cement concrete.

Maine.—Considerable snowplow damage to thermoplastic on bituminous pavement; therefore not economically feasible in heavy snowplow areas.

Maryland.—After 18 months, thermoplastic lost considerable night visibility. In poor condition on portland cement concrete after 2 years. Condition good on bituminous pavement after 8 years at one location.

Michigan.—Better adhesion of thermoplastic on older portland cement concrete and on bituminous pavement. Needed early replacement on new section of portland cement concrete. Not recommended on portland cement concrete. Still usable after 8 years on bituminous pavement.

Minnesota.—Snowplow causes extensive damage to thermoplastic. Poorer adhesion to portland cement concrete may have been caused by application over existing paint stripe. Even with epoxy primer over unpainted area, thermoplastic still had approximately same loss in first year as installations with rubber primer and over paint. For future, recommend light grinding or sandblasting of portland cement concrete, improved epoxy primer, and increased rate of primer application and primer not applied more than 30 minutes before thermoplastic application. Thermoplastic justified on bituminous pavement; still excellent after 2 years. Epoxy applied to portland cement concrete and bituminous pavement at 4,000 linear feet per gallon. When rate increased to 1,000 ft. per gallon thermoplastic began to slide on bituminous pavement but held fast on portland cement concrete.

At installation near St. Paul-Minneapolis, in 1967, 15 percent loss in less than 1 year. Surface preparation of light grinding or sandblasting and increased primer application rate produced little improvement in adhesion to portland cement concrete. Very good performance on bituminous pavement. Performance on portland cement concrete variable between projects, also within a single project and even between adjacent lanes. Leading edge first to deteriorate mainly because of snowplows. Deterioration related to the number of snowplow operations. More failures noted when thermoplastic is placed over existing paint than when no paint previously existed.

Mississippi.—Thermoplastic more effective and cheaper than paint on road with high-traffic density over a 7-year period. Pliobond applied at 6–8 gallons per mile of actual stripe.

Nebraska.—Thermoplastic satisfactory at night where 40 percent or more of line still intact. Daylight appearance not quite up to new paint but performance satisfactory. By comparison, rubber-based primer seems to give better adhesion than epoxy.[5] Some damage by snowplows.

New Hampshire.—Good visibility of thermoplastic gave improved safety compared to paint and its associated degradation. Extensive cracking of thermoplastic owing to cold weather but performance not adversely affected. Minor damage by snowplows on several sections. Paint contract at 2 cents per linear foot is real. Present price now about 3–4 cents per linear foot. For thermoplastic, 19 cents per linear foot is real but low—this was an early installation, perhaps done at cost.

New Jersey.—Reflectivity of thermoplastic decreases with age. In 4 years, project terminated because all lane lines were worn off at curves and were therefore painted.

New Mexico—Thermoplastic may have lasted longer if surface had been sandblasted. No primer and surface preparation used at manufacturer's recommendation.

New York.—Thermoplastic discolors after 4–6 years. Some pinholing and blistering. Blistering over portland cement concrete but not specifically noted over bituminous. Damage to leading edge of lane stripe. Suggest feathering leading edge. Much better adhesion to bituminous concrete than to portland cement concrete. On bituminous concrete, no difference in 1½ years whether placed over synthetic rubber or no primer. Adhesion failures in Long Island installations greatly reduced by use of epoxy primer on portland cement concrete.

After 5 years, thermoplastic still approximately as bright as fresh paint. Thermoplastic more economical than paint on bituminous pavement with high traffic density. Synthetic rubber primer on portland cement concrete gives variable service—less than 1 year on new portland cement concrete to 50 percent retained in 2½ years on 2-year-old portland cement concrete. On new portland cement concrete with epoxy primer, thermoplastic appears satisfactory after 1 year. Thermoplastic seems more visible than paint under wet conditions.

Epoxy primer for portland cement concrete usually applied at 5–7 mils wet film (more recently 4–5 mils) and approximately 15 minutes before thermoplastic. An infrared heater may be used for shorter cure time. Synthetic rubber primer for bituminous pavement contains 10 percent solids, applied at 100 linear feet (6-inch stripe) per gallon and allowed to become tacky before thermoplastic applied (sometimes more than 1 day before thermoplastic application).

Thermoplastic detached from portland cement concrete had thin laitance layer of portland cement concrete on bottom surface. Adhesion losses less if snowplow shoes used rather than no shoes. Edge stripes [illegible] should contain gaps or channels.

Long Island Parkways (SSP 62–2, MSP 62–3, HSPM 63–2, F118 64–1, NSP 65–2): Generally, precleaning for skip line required air blower, whereas edge line required mechanical and hand brooming and blowing, depending on accumulation of debris. Maximum thickness of epoxy 6 mils. Amount of thermoplastic blistering decreased as thickness of epoxy decreased.

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[5] This observation is contrary to reports by other agencies.

Specifications for chemical composition of binder recently changed to reduce bubbling of thermoplastic. Wet film of epoxy binder reduced to 4.5 mils to help prevent bubbling and to better cure epoxy prior to application of thermoplastic. Allow approximately 15 minutes for epoxy to cure before thermoplastic application. Infrared heater may be used to shorten epoxy cure time.

Ohio.—Thermoplastic failed in miles on portland cement concrete, not on bituminous pavement. Snowplows did not particularly disturb material.

Oklahoma.—Thermoplastic is favorable for highways with high traffic density. Faded significantly on Interstate 440—don't know why. Perhaps poor binder. Gores needed painting after 6 years of thermoplastic use. Failure mostly on curves, ramps, and on bridge decks sanded during ice storms. Bituminous pavement, 4–5 years old, needed resurfacing and therefore obliterated thermoplastic still in place.

Oregon.—Thermoplastic lasts longer when not applied over built-up paint layer on older bituminous pavement. Therefore, should remove old layers of paint by sandblasting. Better service when applied over paint film only 1-year-old. On U.S. 20, 40 percent of thermoplastic removed by snowplows—therefore not suitable for mountain passes.

Rhode Island.—Excellent performance over bituminous pavement. Avoid application over bituminous seams, which later crack and affect lane lines.

South Carolina.—Synthetic rubber primer applied at 50 square feet per gallon. Thermoplastic still in good condition, but extrapolating cost of thermoplastic and paint it would take 28 to 32 years of maintenance painting to overcome initial cost of thermoplastic. Therefore thermoplastic not economical under these conditions. On bituminous pavement paint more visible than thermoplastic on rainy day or on dry night, but on portland cement concrete thermoplastic more visible than paint on rainy day.

Texas.—Excellent durability of thermoplastic on bituminous pavement overlay on portland cement concrete, but average life was 4 years on portland cement concrete. Catatherm yellowed considerably while Perma-Line did better. Flaking action noted in winter.

Wisconsin.—On bituminous pavement, reflectance decreased in 1 year. On portland cement concrete, winter plowing destroyed some sections with more than 10 percent failure in first year. Reflectance low after 10 months probably contained less exposed beads than regular paint. Believe portland cement concrete should be sandblasted or acid etched, as thermoplastic performance is unreliable on portland cement concrete. Blisters (cause unknown) evident after 1 week and then break. This perhaps presents rough surface for snowplows to catch and destroy. Where thermoplastic adhered properly, it goes thru winter better than paint and has good daytime appearance. Snowplows catch leading edge of dashed stripe. In view of blister formation in recent replacements on portland cement concrete, relegate thermoplastic use to bituminous surfaces or to only experimental use on portland cement concrete. Originally no difference found whether thermoplastic placed over well adhered paint or over unpainted bare portland cement concrete pavement. Later found better adhesion on new portland cement concrete without paint than on older pavement with paint. On Interstate 94 all thermoplastic lane lines replaced in 1 year. Pavement sandblasted and epoxy-primed, let dry over weekend, and epoxy-primed again immediately prior to thermoplastic application. Blistering occurred again in 1 week. Inconsistent results on different portland cement concrete sections.

Wyoming.—Good results with thermoplastic on bituminous pavement—durability ratio of at least 10:1 over paint on city street and 4:1 on highway. Consideration given to further use.

District of Columbia.—Thermoplastic life on bituminous pavement equal to eight or more paint applications. Plan to use for crosswalks whenever feasible.

Puerto Rico.—Better adhesion on bituminous pavement. Use bonding agent over portland cement concrete and apply thermoplastic immediately over primer. Not recommended for secondary roads, with low traffic volume, requiring painting only once a year.

Unsatisfactory results in December 1963 over portland cement concrete with epoxy primer. Replaced in January 1964 using Pliobond primer. In August 1967 only 30 percent of stripe intact, needed immediate repainting. Bonding is still a problem on portland cement concrete. New installations on portland cement concrete, Perma-Seal II epoxy primer used but too new to determine results.

Toll roads and bridges

Illinois State Toll Highway Commission.—Applied as a rumble stripe in advance of toll plaza. Subject to heavy snowplow activity. No experience with lane and edge lines.

New York Thruway Authority.—Thermoplastic gore markings began to fail within weeks, and in several months, thermoplastic was virtually gone. On lane lines, excellent to beginning of first winter and then plows completely removed it.

Port of New York Authority.—Generally use thermoplastic for lane lines if costs are equal or slightly higher than paint. Smaller additional cost justified by reduced interruptions and delays to traffic.

Triborough Bridge and Tunnel Authority.—Painted line is more economical but thermoplastic line is more effective.

Cities and Counties

Atlanta, Ga.—On bituminous surface, cost of thermoplastic on crosswalk are fully recovered and justified. Usually get eight times life of paint with thermoplastic. Don't use thermoplastic for center and lane lines except on streets with high traffic density.

Baltimore, Md.—Thermoplastic gives superior performance compared to paint. Center and lane lines provide 2–3 year durability where paint lasts less than 1 year. Use thermoplastic for center and lane lines for downtown, expressways, and freeways.

Detroit, Mich.—(Chrysler Freeway—also see Michigan State Highway Department report)—On bituminous pavement, thermoplastic in good condition after 2½ years. On portland cement concrete, thermoplastic poor compared to performance on bituminous pavement. Initial chipping after 3 months, loss increased progressively. Generally starts at leading edge, caused by plows. In 3½ years on portland cement concrete, chipping and failing badly. Costwise, satisfactory in bituminous pavement but poor on portland cement—major repairs needed in 18 months. Need improvement for applications on portland cement concrete.

Los Angeles County.—Ordinarily use hand brooming to clean, grinding to remove old paint, or sandblast to remove curing agent on fresh portland cement concrete. Yellow thermoplastic showed excessive color deterioration, fading and inability to remain self-cleaning in relatively dry climate. Thermoplastic requires life of 6–8 years to compare economically with paint. However, because of road maintenance program, permit work and utilities repair, useful life of thermoplastic is impractical beyond 4 years. With 4 years wear, appearance of thermoplastic is generally poor.

New York, N.Y.—Some damage of thermoplastic by snowplows but not significant. Costwise, thermoplastic cheaper than paint in long run. Durability of thermoplastic over paint is approximately 6:1. Thermoplastic also exhibits safety advantages over paint.

Portland, Oreg.—Thermoplastic should be applied over newly surfaced roadways to get assurance of maximum life. Plan to use more thermoplastic on high-traffic-volume streets.

San Francisco, Calif.—Thermoplastic crosswalks cheaper than paint on medium- and high-traffic-density streets. Work is done in conjunction with resurfacing program.

NEW PUBLICATIONS

The Bureau of Public Roads has recently published two documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

A Study of Airspace Utilization

A Study of Airspace Utilization (75 cents a copy) deals with the general question of airspace utilization over and under freeways. The publication was prepared as the final report of a research study to provide policy and procedure guidelines for the State of California Legislature, the California Division of Highways, the U.S. Bureau of Public Roads, city and county governments and others interested in air rights. The objectives of the study were to identify the major issues and problems connected with freeway air rights; to analyze these issues, including the procedural, legal, technical, financial, aesthetic, and policy aspects of air rights; and to recommend guidelines and design a course of action for the utilization of airspace in California.

Such questions as, "Why are these rights significant," "what uses are desirable" and "under what circumstances will the use of freeway airspace be successful," are answered in terms of the impact upon local communities, the California Division of Highways, the Bureau of Public Roads, and the airspace developer.

Highway Statistics, 1967

Highway Statistics, 1967 ($1.75 a copy) is the 23d issue of the annual compilation of statistical and analytical tabular matter pertaining to Federal aid for highways. This 196-page publication presents information, primarily in tabular form, on motor fuel, motor vehicles, driver licensing, highway-user taxation, State and local highway financing, road and street mileage, and Federal aid for highways.

U.S. GOVERNMENT PRINTING OFFICE: 1969

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–34 are available upon request addressed to Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. 20591.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1966). 20 cents.

Capacity Analysis Techniques for Design of Signalized Intersections (Reprint of August and October 1967 issues of PUBLIC ROADS, a Journal of Highway Research). 45 cents.

Construction Safety Requirements, Federal Highway Projects (1967). 50 cents.

Corrugated Metal Pipe Culverts (1966). 25 cents.

Creating, Organizing, & Reporting Highway Needs Studies (Highway Planning Technical Report No. 1) (1963). 15 cents.

Federal-Aid Highway Map (42 x 65 inches) (1965). $1.50.

Federal Laws, Regulations, and Other Material Relating to Highways (1965). $1.50.

Federal Role in Highway Safety, House Document No. 93, 86th Cong., 1st sess. (1959). 60 cents.

Freeways to Urban Development, A new concept for joint development (1966). 15 cents.

Guidelines for Trip Generation Analysis (1967). 65 cents.

Handbook on Highway Safety Design and Operating Practices (1968). 40 cents.

Highway Beautification Program. Senate Document No. 6, 90th Cong., 1st sess. (1967). 25 cents.

Highway Condemnation Law and Litigation in the United States (1968):
  Vol. 1—A Survey and Critique. 70 cents.
  Vol. 2—State by State Statistical Summary of Reported Highway Condemnation Cases from 1946 through 1961. $1.75.

Highway Cost Allocation Study: Supplementary Report, House Document No. 124, 89th Cong., 1st sess. (1965). $1.00.

Highway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Research and Development Studies. Using Federal-Aid Research and Planning Funds (1967). $1.00.

Highway Statistics (published annually since 1945):
  1965, $1.00; 1966, $1.25; 1967, $1.75.
  (Other years out of print.)

Highway Statistics, Summary to 1965 (1967). $1.25.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Human Values (Annual Report for Bureau of Public Roads) (1966). 75 cents.
  Supplement (1966). 25 cents.

Highways to Beauty (1966). 20 cents.

Highways and Economic and Social Changes (1964). $1.25.

Hydraulic Engineering Circulars:
  No. 5—Hydraulic Charts for the Selection of Highway Culverts (1965). 45 cents.
  No. 10—Capacity Charts for the Hydraulic Design of Highway Culverts (1965). 65 cents.
  No. 11—Use of Riprap for Bank Protection (1967). 40 cents.

Hydraulic Design Series:
  No. 2—Peak Rates of Runoff From Small Watersheds (1961). 30 cents.
  No. 3—Design Charts for Open-Channel Flow (1961). 70 cents.
  No. 4—Design of Roadside Drainage Channels (1965). 40 cents.

Identification of Rock Types (revised edition, 1960). 20 cents.
  Request from Bureau of Public Roads. Appendix, 70 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42, 89th Cong., 1st sess. (1965). 20 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal Aid Construction, 2d ed. (1965). $1.75.
  Amendment No. 1 to above (1966). $1.00.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Sheets and Highways (1961). $2.00.
  Part V only of above—Traffic Controls for Highway Construction and Maintenance Operations (1961). 25 cents.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354, 88th Cong. 2d sess. (1964). 45 cents.

Modal Split—Documentation of Nine Methods for Estimating Transit Usage (1966). 70 cents.

National Driver Register. A State Driver Records Exchange Service (1967). 25 cents.

Overtaking and Passing on Two-Lane Rural Highways—a Literature Review (1967). 20 cents.

Presplitting, A Controlled Blasting Technique for Rock Cuts (1966). 30 cents.

Proposed Program for Scenic Roads & Parkways (prepared for the President's Council on Recreation and Natural Beauty), 1966. $2.75.

Reinforced Concrete Bridge Members—Ultimate Design (1966). 35 cents.

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The Role of Third Structure Taxes in the Highway User Tax Family (1968). $2.25.

Standard Alphabets for Highway Signs (1966). 30 cents.

Standard Land Use Coding Manual (1965). 50 cents.

Standard Plans for Highway Bridges:
  Vol. I—Concrete Superstructures (1968). $1.25.
  Vol. II—Structural Steel Superstructures (1968). $1.00.
  Vol. IV—Typical Continuous Bridges (1962). $1.00.
  Vol. V—Typical Pedestrian Bridges (1962). $1.75.

Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.

Study of Airspace Utilization (1968). 75 cents.

Traffic Assignment Manual (1964). $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Typical Plans for Retaining Walls (1967). 45 cents.

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FEDERAL HIGHWAY ADMINISTRATION
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COVER

Interchange between Interstate Highway 485 (entering from left) and Interstate Highway 75 in Atlanta, Ga. A part of the Atlanta business district is visible in the background.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 16, 1966.

Contents of this publication may be reprinted. Mention of source is requested.

Comparison of Day and Night Gap-Acceptance Probabilities

Reported by [1] NICHOLAS G. TSONGOS, Highway Research Engineer, and SIDNEY WEINER, Mathematician, Traffic Systems Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

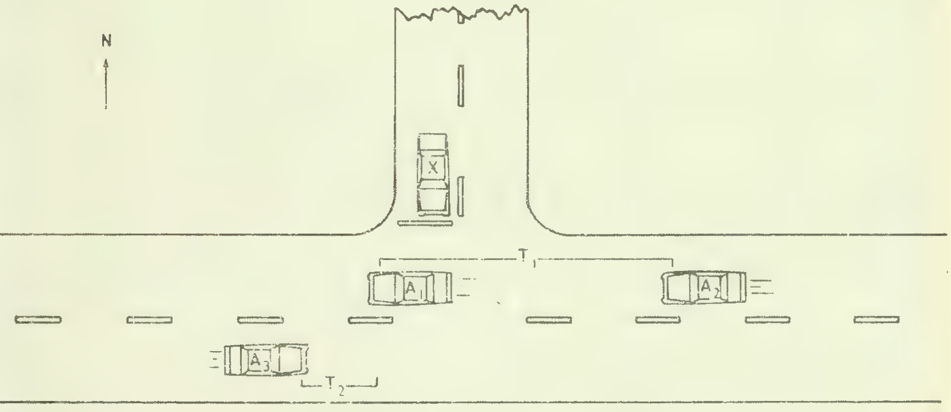
Introduction

THE STUDY reported here was a part of a project to explore the effects of headlight glare from opposing vehicles on the ability of drivers to perform nighttime highway visual tasks. Although a considerable number of studies have been concerned with descriptions of traffic flow at intersections, and especially with gap acceptance and rejection, a search of the literature indicates that no material about the behavior of drivers entering a major roadway from a stopped position at night has been published. The presence of opposing vehicle headlights is one of the most important factors that influence and determine the ability of the driver to see and perform in the night driving situation. The purpose of the study was not to examine the headlamp effect itself, but rather, to determine and compare the headway distributions and gap acceptance between day and night, as a result of the change in ambient illumination available to the driver at night.

Gap-Lag Acceptance, Definitions and Assumptions

- *Gap* is the elapsed time between the arrivals of successive main-street vehicles at an intersection. For minor-street vehicles entering the main street and turning right, conflicting with one traffic stream, the gap is formed by main-street vehicles traveling in the same direction. For minor-street vehicles entering the main street and turning left, conflicting with both traffic streams, the successive vehicle can be traveling either in the same direction or in the opposite direction.
- *Lag* is the elapsed time between the arrival of a minor-street vehicle at an intersection and the arrival of the next main-street car at the intersection.
- *Arrival* of a main-street vehicle at the intersection is the time at which the car enters the area bounded by two pneumatic tubes, installed for the study, at the extensions of the minor-street curb lines. *Arrival* of a minor-street vehicle at the intersection is the time

GAP—AS IT WAS USED IN THE STUDY



Elapsed time (T_1) between arrivals of vehicles A_1 and A_2
or (T_2) between arrivals of vehicles A_1 and A_3

Vehicle X on the side, or minor, street will use a gap, T_1, T_2, or $T_{etc.}$, to enter the traffic stream on the main street. If it uses an oncoming gap to enter the main-street traffic, the gap is accepted. If it waits at the intersection as the gap passes by, the gap is rejected.

Of the many factors that influence the ability of the driver to see and perform the driving task at night, one of the most important is the effect of the headlight beams of other vehicles. In the study reported here, the headway distribution and gap-acceptance probabilities between day and night on an isolated, unlighted, suburban intersection were compared, and the driver's behavior as a result of the change in lighting conditions explored. The test for conditional homogeneity, an analytical method, and Raff's method, a purely graphical method, were used to analyze and compare the data obtained. Although no significant difference in the gap-acceptance probabilities were shown by the Raff method, the results obtained by the analytical method indicated that the gap-acceptance probabilities for day and night cannot be considered conditionally homogeneous, particularly for very short and long gaps. For the medium size gaps, the acceptance probabilities did not indicate significant differences.

It was shown in the study that there were no significant differences between day and night in the formation of the available gaps under light traffic-volume conditions; but as the volume increased, there was a higher percentage of longer gaps at night than during the day.

According to the authors, further experimentation will be necessary before it can be decided that the gap acceptance depends on lighting conditions.

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[1] Presented informally to the Visibility Committee of the Highway Research Board at the 48th annual meeting, Washington, D.C., January 1969.

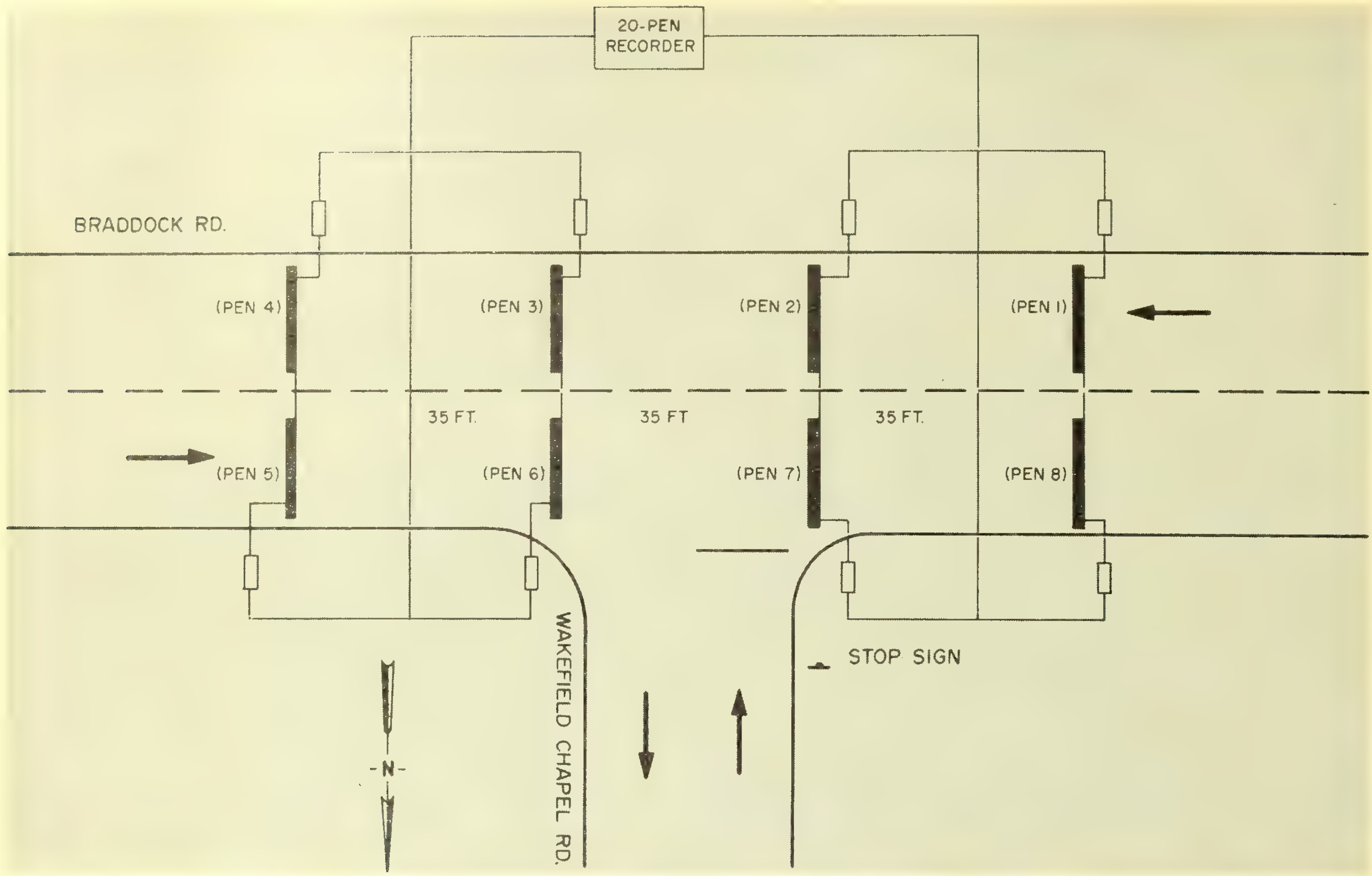

Figure 1.—Layout of detection tubes.

at which the car stops at the stop sign. If a minor-street car under consideration has to stop behind another car waiting to enter the intersection, its arrival is not the time that it stops, but the time that the other car, immediately ahead, enters the intersection.

- A gap is accepted if the minor-street, or side-street, vehicle crosses or enters between two main-street vehicles that form a gap. As a driver can reject any number of gaps but accept only one gap each time that he enters the intersection, the percentage obtained by using all the gaps rejected by each driver would not give a true indication of the proportion of drivers accepting gaps of a certain size, and the conclusions would be inaccurate. To overcome this inaccuracy, the number of rejected gaps must be limited to one for each driver. Accordingly, two assumptions were made as follows: (1) A driver who accepts a given-size gap for the same conditions can be expected to accept any gap of greater length, and (2) a driver who rejects a given-size gap can also be expected to reject all gaps of shorter length. ([illegible] instances in which a driver accepted a shorter gap than he previously had rejected [illegible] in a total of more than 1,200 gap and lag acceptances, day and night.)

- If the minor-street driver enters the intersection before the next main-street car reaches it, the lag is accepted. If he waits until the main-street car has passed before entering the intersection, the lag is rejected.

- The concept of critical gap–lag was used as it was defined by M. S. Raff (*1*)[2] which is "the number of accepted lags shorter than the critical time lag is equal to the number of rejected lags longer than this specific value." In this article, the data for both gaps and lags have been combined and are referred to as gap–lag data.

Test Site and Collection of Data

The test site selected for the study was an isolated, suburban *T* type intersection of State roads (SR) 620 and 650 in Fairfax County, Va. Braddock Road, SR 620, a major 2-lane road, regularly attains a daily traffic volume of 16,000 vehicles and has an early evening hourly volume of 800–1,000 vehicles. Wakefield Chapel Road, SR 650, a minor 2-lane road regulated by a stop sign, attains a daily traffic volume of 3,000 vehicles and has an early evening hourly volume of 200–300 vehicles. At the site, both roads have unrestricted sight distances.

The traffic volume of the intersection, which is 1.35 miles from the nearest traffic signal, does not approach conditions of congestion, and consequently, the times of arrival can be considered to be independent.

Speeds of the main-street vehicles were 30–40 m.p.h. A graphic recorder with a chart speed of 9 in. per min., or 0.15 in. per sec., was wired to air switches and recorded instantly the passage of each vehicle on Braddock Road. The layout of the detection tubes that actuated the air switches is shown in figure 1. These pneumatic tubes were spaced 35 feet apart on Braddock Road at the ends of the intersection curve to avoid false actuations by the turning vehicles.

A dead area of 4 feet—2 feet on either side of the Braddock Road lane line—was provided between adjacent lanes. The line area was wide enough to permit detection of all vehicles in the lane, and small enough to eliminate false actuations by vehicles traveling in adjacent lanes. As each vehicle was detected at the various locations on Braddock Road, the corresponding pen was actuated to make a characteristic mark on the moving tape. Three pens, actuated manually by three pushbuttons, were used to record the vehicles entering Braddock Road. Whenever a minor-street vehicle stopped at the stop sign, the middle pen was moved out of its normal position and held as long as the vehicle was rejecting the available gaps. As soon as the vehicle entered Braddock Road, the middle pushbutton was

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[2] Italic numbers in parentheses refer to the bibliography listed on page 165.

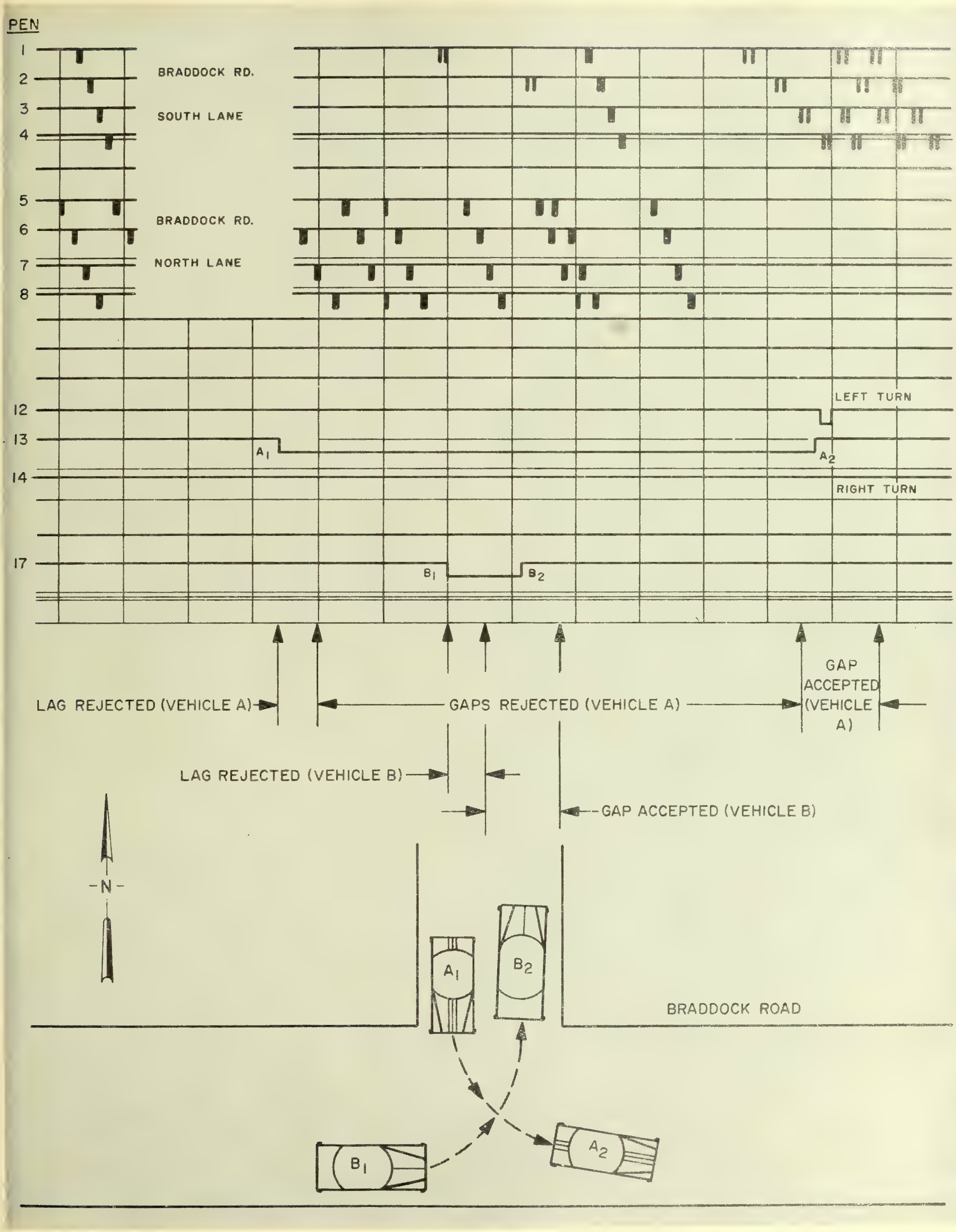

Figure 2.—Data collection, measurements of lags, gaps, and waits.

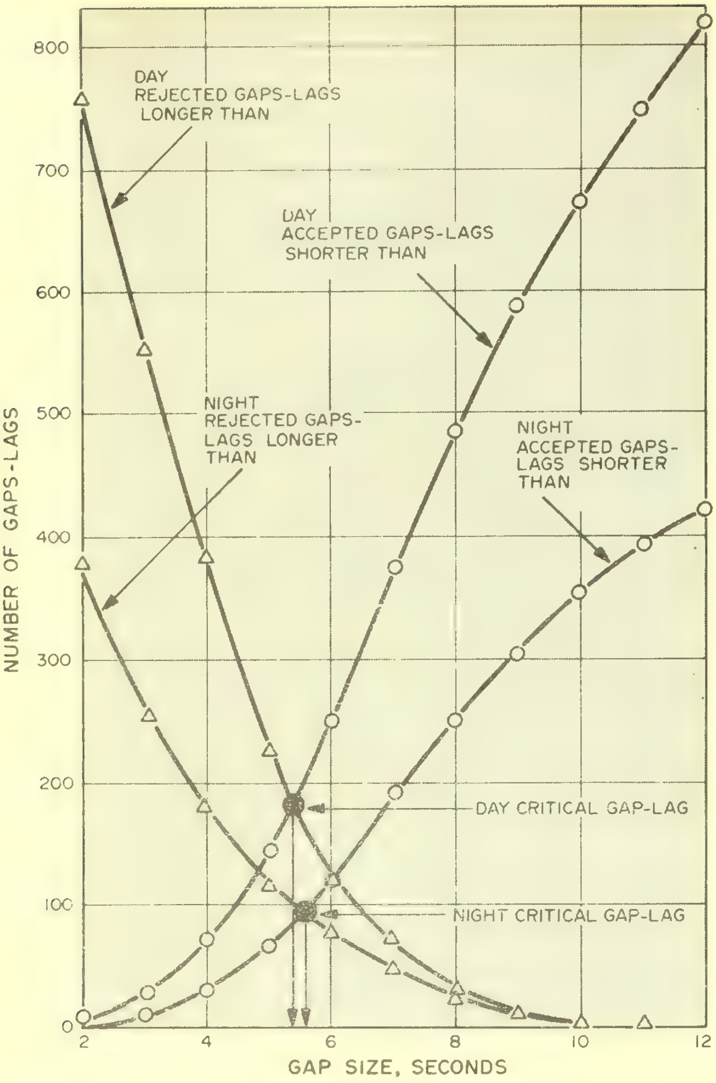

Figure 3.—Distribution of accepted and rejected gaps and lags, all turning movements.

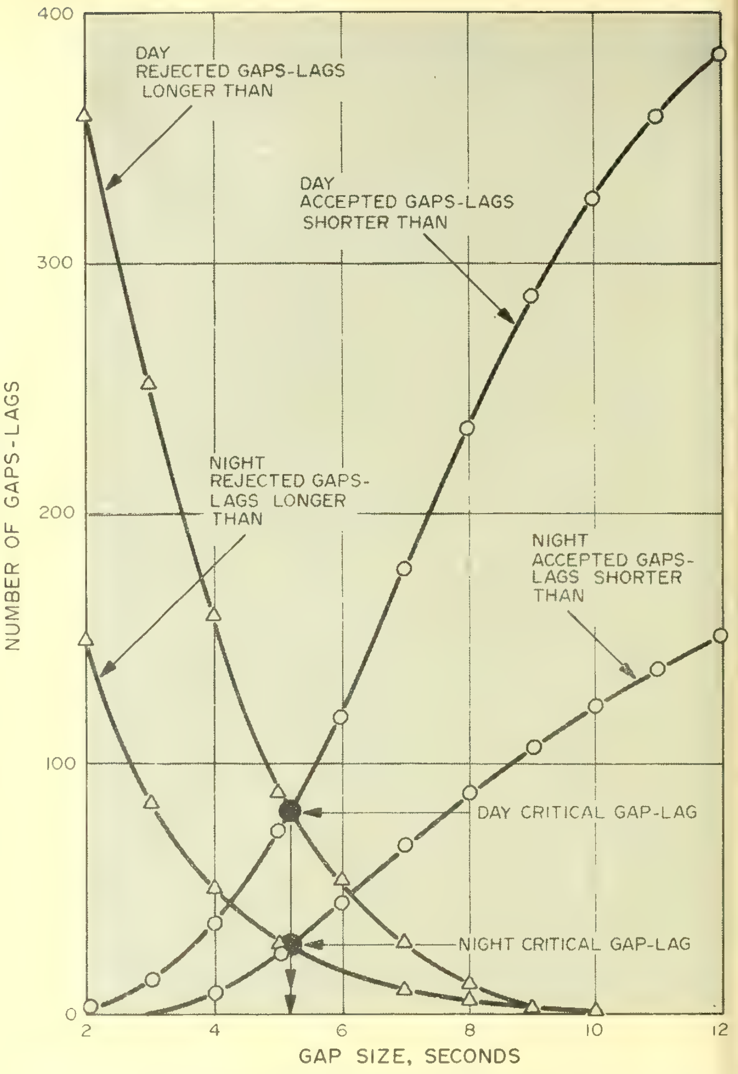

Figure 4.—Distribution of accepted and rejected gaps and lags, conflict with one traffic stream.

released; the pen consequently returned to the normal position, and the appropriate left or right pushbutton was given a quick tap to correspond with the respective left or right maneuver of the vehicle. For main-road vehicles making left turns into Wakefield Chapel Road, the same procedure was followed, except that only one pen showed the time that the vehicle was rejecting the available gaps.

The data collection and recording techniques are shown in figure 2. The role of each pen is indicated on a sample section of the chart, and the intersection drawing illustrates the position of the different vehicles as they are recorded on the chart.

Methodology

Test for conditional homogeneity

In the analysis it was assumed that each driver's decision to accept or reject a gap size is mutually independent and that the probability of [illegible] constant. Because of the [illegible] of gap observations, the analysis was conducted on the joint set of observations of random gap and lag sizes. It was then further assumed that the driver's probability of accepting a given gap size is equal to that for the same lag size.

The independent random sets of gap lag acceptances and rejections for daytime and nighttime lead to a series of sampled proportions as follows:

Gap–lag size, seconds:

	1	2	3		c
Day	$\hat{P}_{11}$	$\hat{P}_{12}$	$\hat{P}_{13}$	. . .	$\hat{P}_{1c}$
Night	$\hat{P}_{21}$	$\hat{P}_{22}$	$\hat{P}_{23}$	. . .	$\hat{P}_{2c}$

$$\hat{P}_{1j}=\frac{a_j}{a_j+r_j}$$

and

$$\hat{P}_{2j}=\frac{A_j}{A_j+R_j}$$

Where,

a_j= number of acceptances for the j^{th} gap–lag size in the daytime.

r_j= number of rejections for the j^{th} gap–lag size in the daytime.

A_j= number of acceptances for the j^{th} gap–lag size at night.

R_j= number of rejections for the j^{th} gap–lag size at night.

It is hypothesized that if a given gap size is j then the underlying probabilities of gap acceptance for day and night, P_{1j} and P_{2j}, are the same for all j's—$j=1, 2 \ldots$ c. This is the hypothesis of conditional homogeneity of the set of corresponding day and night gap-acceptance probabilities. Thus, if the hypothesis $P_{1j}=P_{2j}$ for all j's is true, it is meant that the day and night samples are conditionally homogeneous as opposed to being strictly homogeneous, which in addition, would be:

$$P_{i1}=P_{i2}= \ldots =P_{ic} \text{ for } i=1, 2$$

In accordance with the procedure and notation given in reference (*2*), the null hypothesis, H_2, is tested against the alternate hypothesis, H_1, where it is defined (for each $j=1,2,3 \ldots c$):

$$H_1: P_{1jk} \neq P_{2jk};$$

$$\sum_{k=1}^{2} P_{ijk} = P_{ij.};\ i=1, 2$$

$$H_2: P_{1jk} = P_{2jk} = P_{\cdot jk};$$

$$\sum_{k=1}^{2} P_{\cdot jk} = P_{\cdot j1} + P_{\cdot j2} = P_{\cdot j}$$

Where,

$$\sum_{k=1}^{2} P_{\cdot jk} = P_{\cdot j}$$

The null hypothesis, H_2, states that there is conditional homogeneity, and H_1 states that the day and night samples have different acceptance $k=1$, and rejection, $k=2$, probabilities among the gap–lag sizes, j.

This procedure is equivalent to a chi-square procedure, involving a set of 2×2 contingency tables. However, the computations here are much less arduous, especially when use is made of table $2n \log_e n$ given in reference (*3*).

The information component for a null hypothesis of conditional homogeneity, H_2, in which:

$$\frac{P_{ijk}}{P_{ij.}} = \frac{P_{\cdot jk}}{P_{\cdot j.}}$$

for

$$i=1, 2;\ j=1, 2, \ldots c;\ k=1,2$$

is

$$2\hat{I}\ (H_1{:}H_2) = 2 \sum_{i=1}^{2} \sum_{j=1}^{c} \sum_{k=1}^{2} X_{ijk} \log \frac{X_{ijk}}{\frac{X_{ij.}X_{\cdot jk}}{X_{\cdot j.}}}$$

Where,

X_{ijk} is the number of observations for the i, j, k category (see table 1), and where,

$i=1$ represents day, $i=2$ represents night

$j=1, 2, \ldots$ c represents gap–lag size category

$k=1$ represents accepted category

$k=2$ represents rejected category

Finally the gap marginal total for each category is given by the expression:

$$X_{\cdot j.} = X_{1j.} + X_{2j.} = X_{\cdot j1} + X_{\cdot j2}$$
$$= X_{1j1} + X_{2j1} + X_{1j2} + X_{2j2}$$

Table 1.—Test of conditional homogeneity between two samples[1]

	Gap size			
	$j=1$	$j=2$		$j=c$
	Seconds	*Seconds*		*Seconds*
Day ($i=1$):				
Accept ($k=1$)	X_{111}	X_{121}		X_{1c1}
Reject ($k=2$)	X_{112}	X_{122}		X_{1c2}
Night ($i=2$):				
Accept ($k=1$)	X_{211}	X_{221}	. . .	X_{2c1}
Reject ($k=2$)	X_{212}	X_{222}		X_{2c2}
MARGINAL TOTALS				
Day ($X_{1j.}=X_{1j1}+X_{1j2}$)	$X_{11.}$	$X_{12.}$	.	$X_{1c.}$
Night ($X_{2j.}=X_{2j1}+X_{2j2}$)	$X_{21.}$	$X_{22.}$	. . .	$X_{2c.}$
Accept ($X_{\cdot j1}=X_{1j1}+X_{2j1}$)	$X_{\cdot 11}$	$X_{\cdot 21}$	. . .	$X_{\cdot c1}$
Reject ($X_{\cdot j2}=X_{1j2}+X_{2j2}$)	$X_{\cdot 12}$	$X_{\cdot 22}$	. . .	$X_{\cdot c2}$
MARGINAL TOTALS				
$X_{\cdot j.}=X_{1j.}+X_{2j.}$ $=X_{\cdot j1}+X_{\cdot j2}$	$X_{\cdot 1.}$	$X_{\cdot 2.}$	. . .	$X_{\cdot c.}$
TEST STATISTIC (DISTRIBUTED AS χ^2 WITH c d.f.)				
$2\hat{I}=2\sum_{i=1}^{2}\sum_{j=1}^{c}\sum_{k=1}^{2} X_{ijk} \ln \frac{X_{ijk}}{\frac{X_{ij.}X_{\cdot jk}}{X_{\cdot j.}}}$				

[1] Sample 1—Gap acceptances and rejections, day. Sample 2—Gap acceptances and rejections, night.

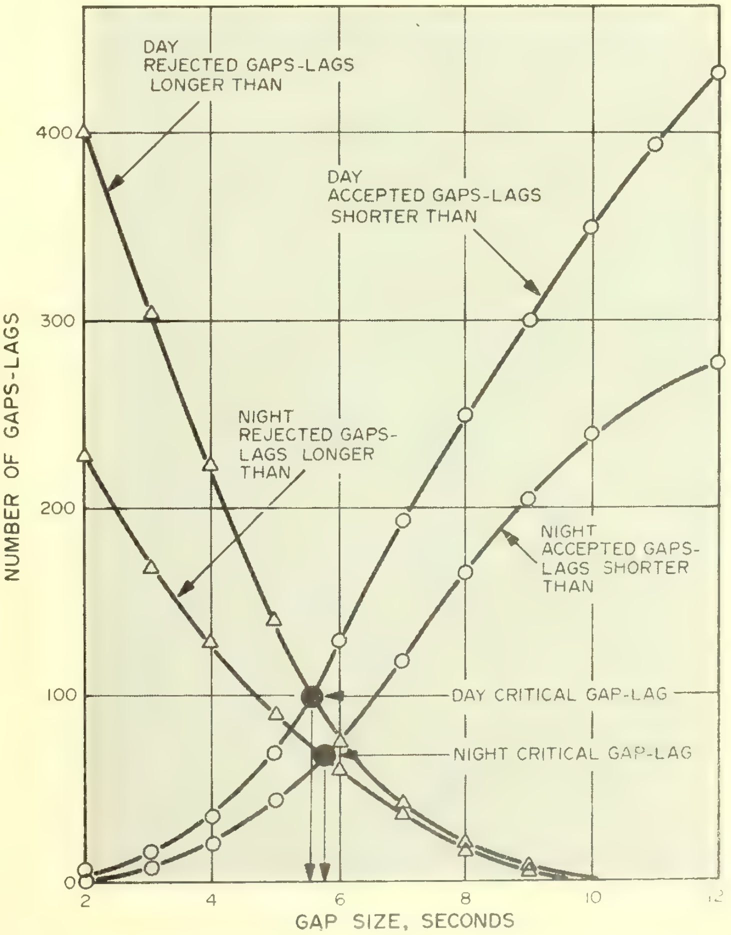

Figure 5.—Distribution of accepted and rejected gaps and lags, conflict with two traffic streams.

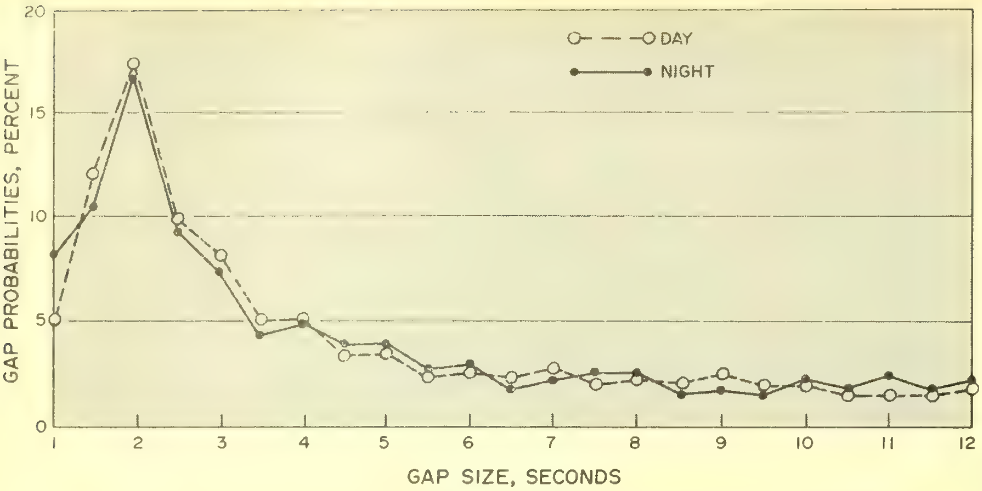

Figure 6.—Gap distribution, Braddock Road, 700–900 vehicles per hour.

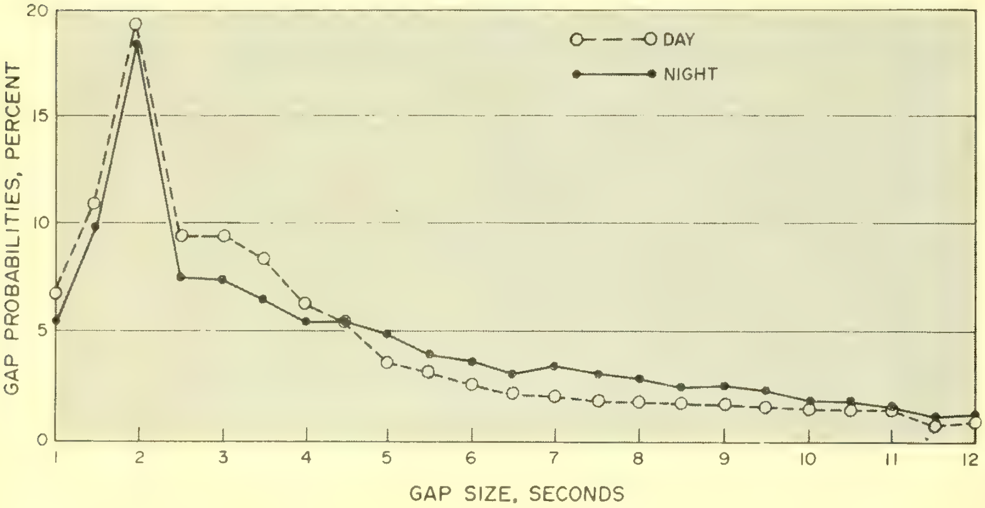

Figure 7.—Gap distribution, Braddock Road, 901–1,100 vehicles per hour.

The statistic $2\hat{I}$ has the asymptotic χ^2 distribution with c degrees of freedom under the null hypothesis of conditional homogeneity. Accordingly, in the second line of table 1, X_{122} represents the occurring number of rejections during the day for gap size $j=2$, and in the third line, X_{221} represents the occurring number of acceptances during the night for gap size $j=2$.

The entry $X_{1j.}$ in the table, under *Marginal totals,* represents the total number of acceptances and rejections during the day for gap size j:

$$\text{Day marginal } X_{1j.} = X_{1j1} + X_{1j2}$$

Similarly under *Marginal totals* are listings for night, accept, and reject:

$$\text{Night marginal } X_{2j.} = X_{2j1} + X_{2j2}$$

$$\text{Accept marginal } X_{.j1} = X_{1j1} + X_{2j1}$$

$$\text{Reject marginal } X_{.j2} = X_{1j2} + X_{2j2}$$

In tables 2, 3, and 4 are summarized the gap–lag acceptance and rejection data for the different movements of the minor-street vehicles—all data, conflicts with one traffic stream, and conflicts with two traffic streams, respectively. The left turn of the minor-street vehicle was defined as *conflict with two traffic streams,* and both the right turn of the minor-street vehicle and the left turn of the major-street vehicle were defined as *conflict with one traffic stream.*

Gap and lag size data were compiled into five groups for each set of data and the hypothesis of conditional homogeneity was tested according to the procedure shown in the bottom section of table 1.

Using the tables in reference (*3*), the statistic $2\hat{I}$ (H_1:H_2) may be easily calculated by expressing it as:

$$2\hat{I} = 2\sum_{i=1}^{2}\sum_{j=1}^{c}\sum_{k=1}^{2} X_{ijk} \ln \frac{X_{ijk}}{(X_{ij.}X_{.jk})} X_{.j.}$$

$$= \sum_{i}\sum_{j}\sum_{k} 2X_{ijk} \ln X_{ijk} + \sum_{j=1}^{c} 2X_{.j.} \ln X_{.j.}$$

$$-\sum_{i=1}^{2}\sum_{j=1}^{c} 2X_{ij.} \ln X_{ij.} - \sum_{j=1}^{c}\sum_{k=1}^{2} 2X_{.jk} \ln X_{.jk}$$

The X_{ijk}'s in this formula are defined in table 1 as the random number of occurrences in the i, j, k category.

As $2n \ln n$ is tabulated in reference (*3*) for [illegible] from 1 to 10,000, the quantities in [illegible] be used as inputs to the tabula- [illegible]

$\sum_{j=1}$ [illegible] $2X_{.j.} \ln X_{.j.}$

[illegible] $\ln X_{.j.}$

Table 2.—Analysis of gap-lag acceptance between day and night—all data

	Gap size				
	$j=1$ (2–3)	$j=2$ (4–5)	$j=3$ (6–7)	$j=4$ (8–9)	$j=5$ (10+ over)
	Seconds	*Seconds*	*Seconds*	*Seconds*	*Seconds*
Day ($i=1$):					
Accept ($k=1$)	25	120	233	210	229
Reject ($k=2$)	377	262	98	24	. . .
Night ($i=2$):					
Accept ($k=1$)	5	62	124	108	128
Reject ($k=2$)	200	108	53	13	6
MARGINAL TOTALS					
Day ($X_{1j.}$)	402	382	331	234	229
Night ($X_{2j.}$)	205	170	177	121	134
Accept ($X_{.j1}$)	30	182	357	318	357
Reject ($X_{.j2}$)	577	370	151	37	6
Gap marginal total ($X_{.j.}$)	607	552	508	355	363

TEST OF CONDITIONAL HOMOGENEITY

$$2\hat{I} = 2\sum_{i=1}^{2}\sum_{j=1}^{5}\sum_{k=1}^{2} X_{ijk} \ln \frac{X_{ijk}\,X_{.j.}}{X_{ij.}\,X_{.jk}}$$

$$= \sum_{i}\sum_{j}\sum_{k} 2X_{ijk} \ln X_{ijk} + \sum_{j=1}^{5} 2X_{.j.} \ln X_{.j.} - \sum_{i}\sum_{j} 2X_{ij.} - \sum_{j}\sum_{k} 2X_{.jk} \ln X_{.jk}$$

$$= 24{,}642.940 + 29{,}528.820 - 26{,}480.349 - 27{,}673.271 = 18.140$$

$$\chi_5^2(.05) = 11.0$$

Raff's critical-lag comparison method

In addition to the technique just described, the method employed by Raff (*1*) was applied to the original data collected in the study to compare the daytime and nighttime gap and lags. Raff determined the critical lag by plotting two cumulative distributions on the same graph, as shown in figures 3, 4, and 5. One curve describes the accepted number of lags that were shorter than a given time interval, and the other shows the rejected number of lags that were longer than this interval. The value of the critical lag was determined as the point at which the two curves intersect on the chart.

Results of the Analysis

It was realized that the traffic volume could affect gap-acceptance probabilities and that if experimental data were collected under varying traffic volume conditions, the true differences between day and night effects could be obscured. Therefore, to substantially negate the traffic volume effect, all the data were divided into two groups according to traffic volume. One group was for a lighter

Table 3.—Analysis of gap-lag acceptance between day and night—conflict with one traffic stream

	Gap size				
	$j=1$ (2–3)	$j=2$ (4–5)	$j=3$ (6–7)	$j=4$ (8–9)	$j=5$ (10 + over)
Day ($i=1$):	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*
Accept ($k=1$)	9	64	105	110	94
Reject ($k=2$)	201	106	42	12	
Night ($i=2$):					
Accept ($k=1$)	...	26	44	36	44
Reject ($k=2$)	97	37	9	4	2

MARGINAL TOTALS

Day (X_{1j})	210	170	147	122	94
Night (X_{2j})	97	63	53	40	46
Accept ($X_{.j1}$)	9	90	149	146	138
Reject ($X_{.j2}$)	298	143	51	16	2
Gap marginal total ($X_{.j}$)	307	233	200	162	140

TEST OF CONDITIONAL HOMOGENEITY

$$2\hat{I}=2\sum_{i=1}^{2}\sum_{j=1}^{5}\sum_{k=1}^{2}X_{ijk}\ln\frac{X_{ijk}X_{.j}}{X_{ij}\,X_{.jk}}$$

$$=\sum_{i}\sum_{j}\sum_{k}2X_{ijk}\ln X_{ijk}+\sum_{j=1}^{5}2X_{.j.}\ln X_{.j.}-\sum_{i}\sum_{j}2X_{ij}-\sum_{j}\sum_{k}2X_{.jk}\ln X_{.jk}$$

$$=9{,}233.211+11{,}207.837-9{,}963.192-10{,}463.211=20{,}441.048-20{,}426.403$$

$$=14.645 \qquad \chi^2_5(.05)=11.0$$

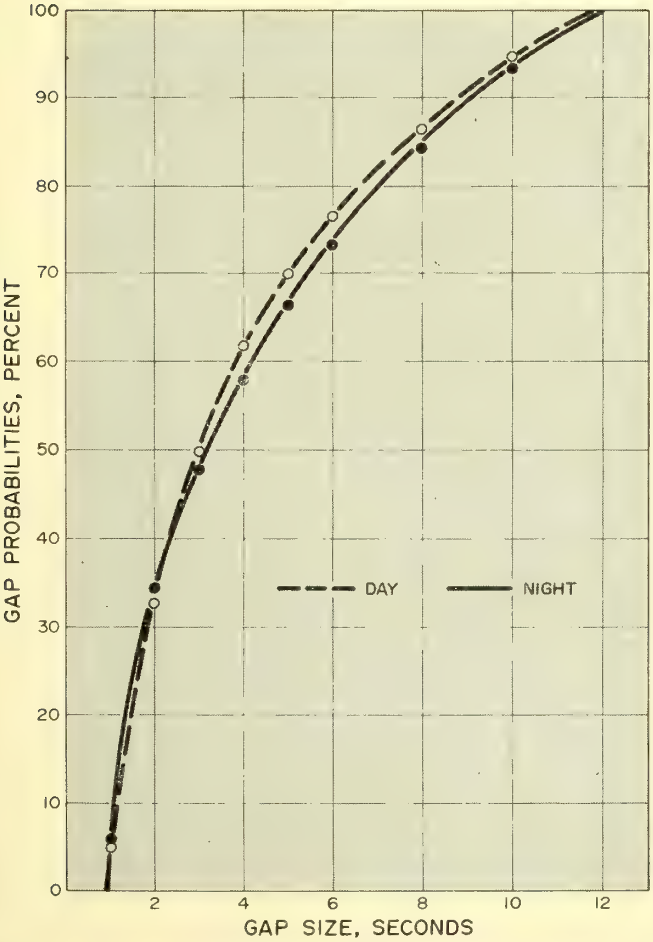

Figure 8.—Cumulative gap distribution, Braddock Road, 700–900 vehicles per hour.

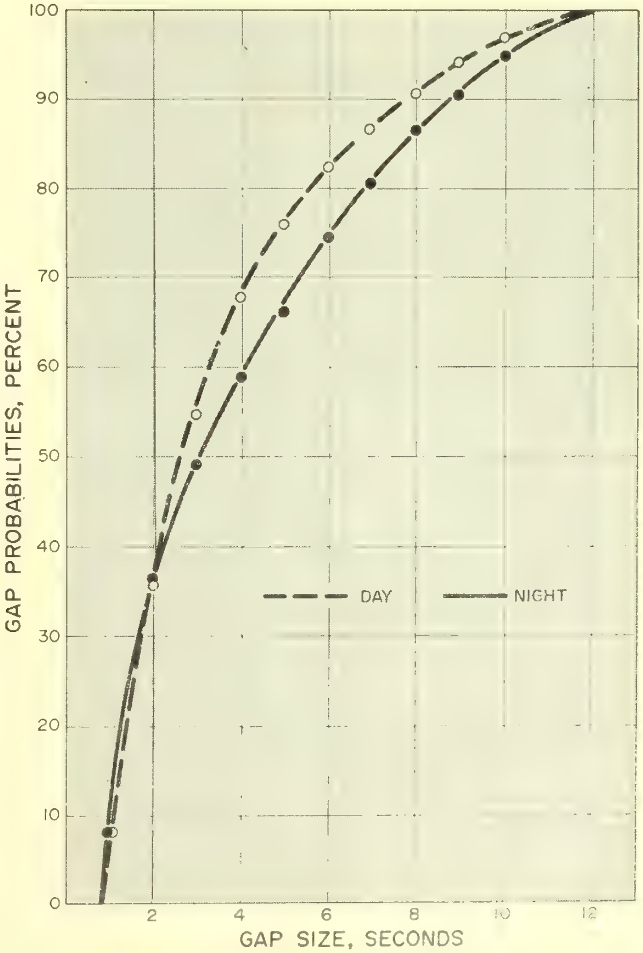

Figure 9.—Cumulative gap distribution, Braddock Road, 901–1,100 vehicles per hour.

Table 4.—Analysis of gap-lag acceptance between day and night—conflict with two traffic streams

	Gap size				
	$j=1$ (2-3)	$j=2$ (4-5)	$j=3$ (6-7)	$j=4$ (8-9)	$j=5$ (10+ over)
	Seconds	*Seconds*	*Seconds*	*Seconds*	*Seconds*
Day ($i=1$)					
Accept ($k=1$)	16	56	128	100	135
Reject ($k=2$)	176	156	56	12	-
Night ($i=2$)					
Accept ($k=1$)	5	36	80	72	84
Reject ($k=2$)	103	71	44	9	4
MARGINAL TOTALS					
Day ($X_{1\cdot\cdot}$)	192	212	184	112	135
Night ($X_{2\cdot\cdot}$)	108	107	124	81	88
Accept ($X_{\cdot\cdot1}$)	21	92	208	172	219
Reject ($X_{\cdot\cdot2}$)	279	227	100	21	4
Gap-lag size total ($X_{\cdot j\cdot}$)	300	319	308	193	223

TEST OF CONDITIONAL HOMOGENEITY

$$2\hat{I}=\sum_{i=1}^{2}\sum_{j=1}^{5}\sum_{k=1}^{2}X_{ijk}\ln\frac{X_{ijk}\,X_{\cdot j\cdot}}{X_{ij\cdot}\,X_{\cdot jk}}$$

$$=\sum_{i}\sum_{j}\sum_{k}2X_{ijk}\ln X_{ijk}+\sum_{j=1}^{5}2X_{\cdot j\cdot}\ln X_{\cdot j\cdot}-\sum_{i}\sum_{j}2X_{ij\cdot}\ln X_{ij\cdot}-\sum_{j}\sum_{k}2X_{\cdot jk}\ln X_{\cdot jk}$$

$=12,212.334+15,073.199-13,297.202-13,976.583=11.748$

$\chi_5^2(.05)=11.0$

Table 5.—Gap availability, Braddock Road—average number of cars per hour for specific traffic volumes [1]

Volume	Day		Night	
	Eastbound	Westbound	Eastbound	Westbound
Vehicles per hr.				
700–900	171	333	103	367
901–1,100	336	328	316	392

[1] Gap size included 1–20 seconds

Table 6.—Gap distribution percentiles, Braddock Road [1]

Percentile	700–900 vehicles per hr.		901–1,100 vehicles per hr.	
	Day	Night	Day	Night
50	3.0	3.2	2.6	3.1
75	5.8	6.1	4.8	6.2
90	8.8	9.2	8.0	9.2

[1] Gap size included 1–12 seconds

Table 7.—Gap-lag size, number of acceptances and rejections—all data

	Seconds											Total
	2	3	4	5	6	7	8	9	10	11	12	
[illegible]	13	12	42	78	104	129	105	105	82	75	72	817
[illegible]	205	172	155	107	50	48	18	6	----		----	761
[illegible]	218	184	197	185	154	177	123	111	82	75	72	1,578
[illegible]		8	16	43	47	77	57	51	61	42	25	427
[illegible]	126	74	62	46	26	27	7	6	6		----	380
[illegible]	126	82	78	89	73	104	64	57	67	42	25	807

traffic volume of 700–900 vehicles per hour, and the other was for a heavier traffic volume of 901–1,100 vehicles per hour. It is considered that each of these groups represented conditions under which gap-acceptance probabilities should be fairly uniform at night or during the day.

In the 700–900 vehicle-per-hour group, a reasonable agreement was evident in the daytime and nighttime gap distribution (figs. 6 and 8). In the 901–1,100 vehicle-per-hour group, the distribution of the larger gaps was somewhat higher during the nighttime (figs. 7 and 9). Some numerical values of the gap distribution in the study intersection are listed in tables 5 and 6, and the results of the collected data are shown in tables 7, 8, and 9.

In the analysis of the test of homogeneity between day and night gap–lag acceptance distribution, it was found that $2\hat{I}=14.645$ and 11.748, respectively, for the groups *conflict with one traffic stream* and *conflict with two traffic streams*. For all the data combined, $2\hat{I}=18.140$. The χ^2 value, with $C(r-1)(r-1)=5$ degrees of freedom and 5 percent level is 11.07. Therefore, statistical differences existed in the overall gap–lag acceptance distribution between the day and night environments at the test intersection.

However, a further exploration was made to detect the size-groups that contributed to the rejection of the hypothesis of homogeneity. The test was applied to each size-group separately, and it was found that groups $j=1$ and 5 had $2\hat{I}$ values greater than the χ^2 value, and that groups $j=2$, 3, and 4 had no significant difference that could be detected for a hypothesis of conditional homogeneity. The $2\hat{I}$ values for each gap size-group, and the corresponding acceptance or rejection of the hypothesis, are shown in table 10.

The corresponding median acceptance-rejection times were very close and, for any practical purpose, can be considered to be the same (table 11). For all the data combined, the median day acceptance times were .03 seconds lower than those at night, whereas the rejection times were .15 seconds higher. A comparison of the two movement-groups showed lower acceptance and rejection times in both day and night gaps and lags for the group *conflict with one traffic stream.* A comparison of the acceptance times between day and night showed that, although the difference is very small, the accepted and rejected times at night were higher and lower respectively for both movement groups.

The Raff method, mentioned previously, was used to obtain the critical gap and lag values. For all the data combined, the critical day gap–lag was 5.4 seconds compared to the night value of 5.6 seconds. This same difference, 0.2 sec., was evident between the day and night critical gap–lag, even though the values were somewhat higher for the group *conflict with two traffic streams.* Aside from the small differences, the resultant values of the Raff method depend largely on the manner in which the curves are plotted on the data points; therefore, this method is a pure visual fitting technique.

Conclusions

The following conclusions, valid only for the intersection under consideration, were inferred from the findings of the reported field investigation:

- There were no significant differences in the formation of gaps between day and night, but it was noticed that, as the volume increased, a higher percentage of long gaps was present at night than during the day.
- Also there were no differences between the median gap–lag acceptance times. The overall median acceptance times for day and night were 7.29 and 7.32 seconds respectively; the overall median rejection times for day and night were 4.01 and 3.86 seconds respectively.
- Night drivers accepted no gap or lag less than 3 seconds and rejected those higher than 10 seconds.
- Day drivers accepted no gap or lag less than 2 seconds and rejected those higher than 9 seconds.
- The overall critical night gap lag, as defined, was 0.2 seconds higher than the day gap–lag.
- The statistical test used was based on the assumption that successive observations are independent in the probability sense. Because the $2\hat{I}$ values were greater than the χ^2 value, the gap acceptance distribution for day and night could not be considered conditionally homogeneous. However this was found to be true only for the very short and very long gaps 2–3 and 10–12 seconds. In the median size gaps, 4–9 seconds, the distribution of the gap acceptances was the same both during the day and at night.
- Further experimentation and analysis is necessary, especially for the short and long size gaps, before it can be determined that the results of day or night acceptance and rejection are dependent on time.

Table 8.—Gap-lag size, number of acceptances and rejections—conflict-with-one-traffic-stream data

	Seconds											Total
	2	3	4	5	6	7	8	9	10	11	12	
Day:												
Accepted	5	4	26	38	44	61	53	57	34	31	29	382
Rejected	109	92	71	35	26	16	10	2				361
Total	114	96	97	73	70	77	63	59	34	31	29	743
Night:												
Accepted			7	19	15	29	17	19	17	14	13	150
Rejected	65	32	24	13	5	4	2	2	2			149
Total	65	32	31	32	20	33	19	21	19		13	299

Table 9.—Gap–lag size, number of acceptances and rejections—conflict-with two-traffic-streams data

	Seconds											Total
	2	3	4	5	6	7	8	9	10	11	12	
Day:												
Accepted	8	8	16	40	60	68	52	48	48	44	43	435
Rejected	96	80	84	72	24	32	8	4				400
Total	104	88	100	112	84	100	60	52	48	44	43	835
Night:												
Accepted		5	12	24	32	48	40	32	44	28	12	277
Rejected	61	42	38	33	31	23	5	4	4			241
Total	61	47	50	57	63	71	45	36	48	28	12	518

Table 10.—Test of the hypothesis of homogeneity for each gap–size group separately

Gap–size group	$2\hat{I}$ Values				
	$j=1$	$j=2$	$j=3$	$j=4$	$j=5$
All data	4.629	1.351	0.007	0.021	12.132
Hypothesis [1]	Rejected	Accepted	Accepted	Accepted	Rejected
Conflict with one traffic stream	6.961	0.025	2.922	0.001	4.508
Hypothesis [1]	Rejected	Accepted	Accepted	Accepted	Rejected
Conflict with two traffic streams	1.546	1.786	0.857	0.007	7.552
Hypothesis [1]	Accepted	Accepted	Accepted	Accepted	Rejected

[1] X^2 (.05) for 1 d.f.=3.84.

Table 11.—Median acceptance–rejection times for gaps and lags combined

Gap size-group	Combined gap–lag median time					
	700–900 vehicles per hr.		901–1,100 vehicles per hr.		All data	
	Day	Night	Day	Night	Day	Night
	Seconds	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*
Group I—Conflict with one traffic stream:						
Accepted	7.04	7.50	7.35	7.21	7.24	7.30
Rejected	3.63	3.42	3.74	3.20	3.78	3.29
Group II—Conflict with two traffic streams:						
Accepted	7.17	7.47	7.47	7.62	7.33	7.53
Rejected	4.32	4.21	4.29	4.24	4.28	4.23
All movements:						
Accepted					7.29	7.32
Rejected					4.01	3.86

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Rapid testing on 6-inch silty-gravel compacted base course. Above left—Plate-bearing test using truck-mounted, hydraulically actuated equipment. Above right—Refraction seismographic test.

Quality Assurance in Highway Construction

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Part 2—
Quality Assurance of Embankments and Base Courses

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This is the second part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, was presented in the previous issue of PUBLIC ROADS. The remaining parts, to be presented in succeeding issues, are 3.—Quality Assurance of Portland Cement Concrete, 4.—Variations of Bituminous Construction, 5.—Summary of Research for Quality Assurance of Aggregate, and 6.—Control Charts.

Introduction

EMBANKMENTS and base courses, essentially, are structural elements of the highway and are amenable to the same treatment as any other structural element with respect to design, process control, and acceptance. Their function is to provide adequate support to the pavement within the design concepts of load applications.

Density Control

The engineer has learned that proper compaction is essential to the performance properties of soil and rock material. However, the uniformity of support is as important, if not more so, than the absolute magnitude of the support offered; therefore, the control of the compaction process is one of the most important aspects in base and embankment construction.

In the 19th Century, during construction of earth dams, it was discovered that the driving of livestock, particularly sheep, across lifts of soil, as they were placed, improved uniformity of support, increased stability, and decreased permeability of the completed structure. Although many improved methods of compaction and compaction control have evolved over the years from this crude beginning, compaction control is still an item of major concern to the highway engineer.

The first attempt toward scientific control of the compaction process resulted from the work of R. R. Proctor (*1*),[1] who developed the moisture-density relations still used in compaction specifications and control. He also developed the Proctor Needle to control the uniformity of compaction. Later the overflow-volumeter, sand-cone, and rubber-balloon methods were developed to aid in the density measurement of compacted materials. The newest, and probably the best methods of measuring moisture and density of compacted materials are those in which nuclear devices are used.

The advent of nuclear equipment not only has provided a faster and better procedure

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[1] Italic numbers in parentheses identify the references listed on page 174.

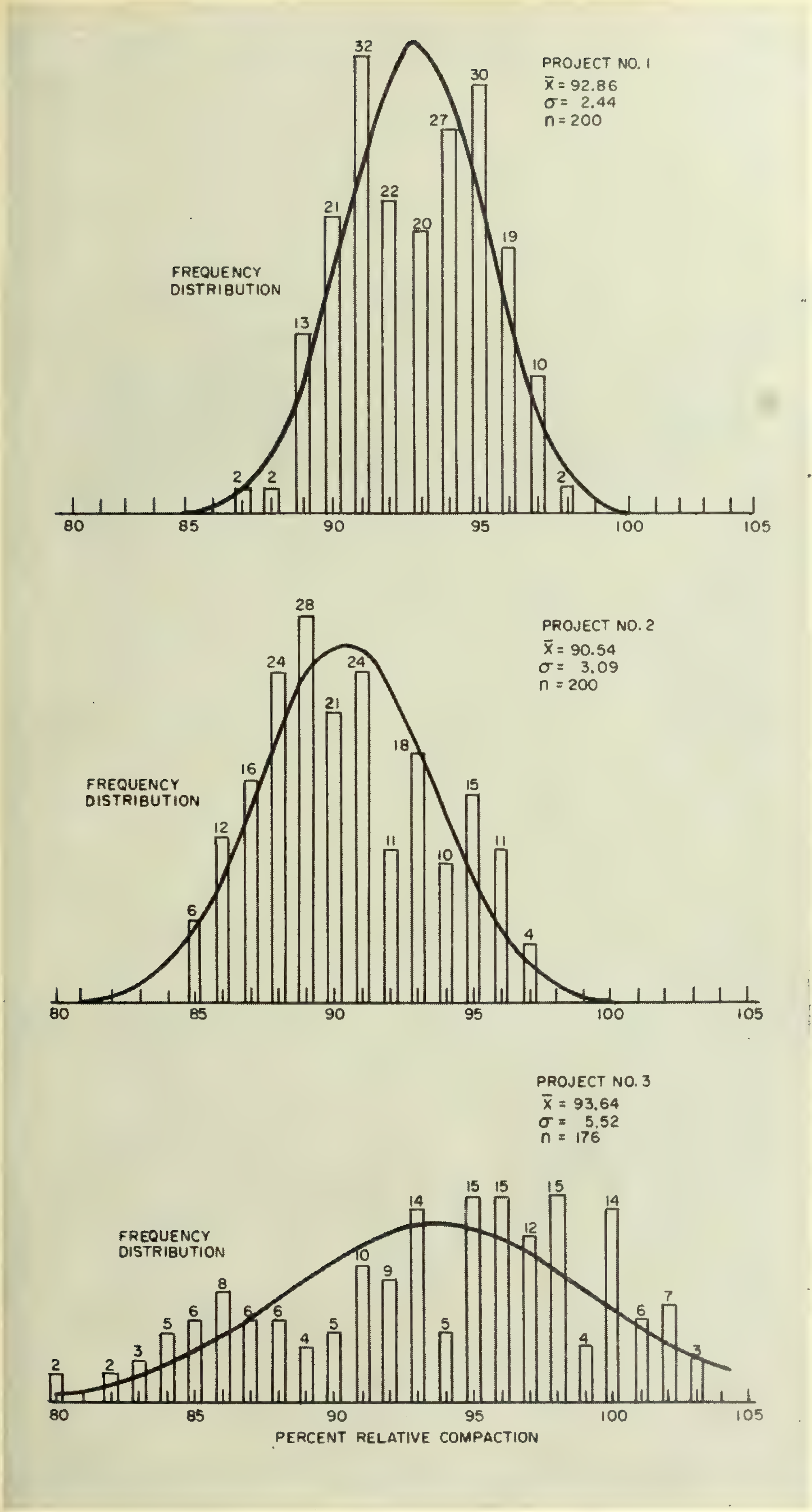


Figure 1.—Variation in density of embankments, California road-embankment study.

for measuring compaction but has resulted in a review of the methods and the precisions to be expected. Also, extensive studies are being made to develop better criteria than density for specifying and controlling compaction in the future.

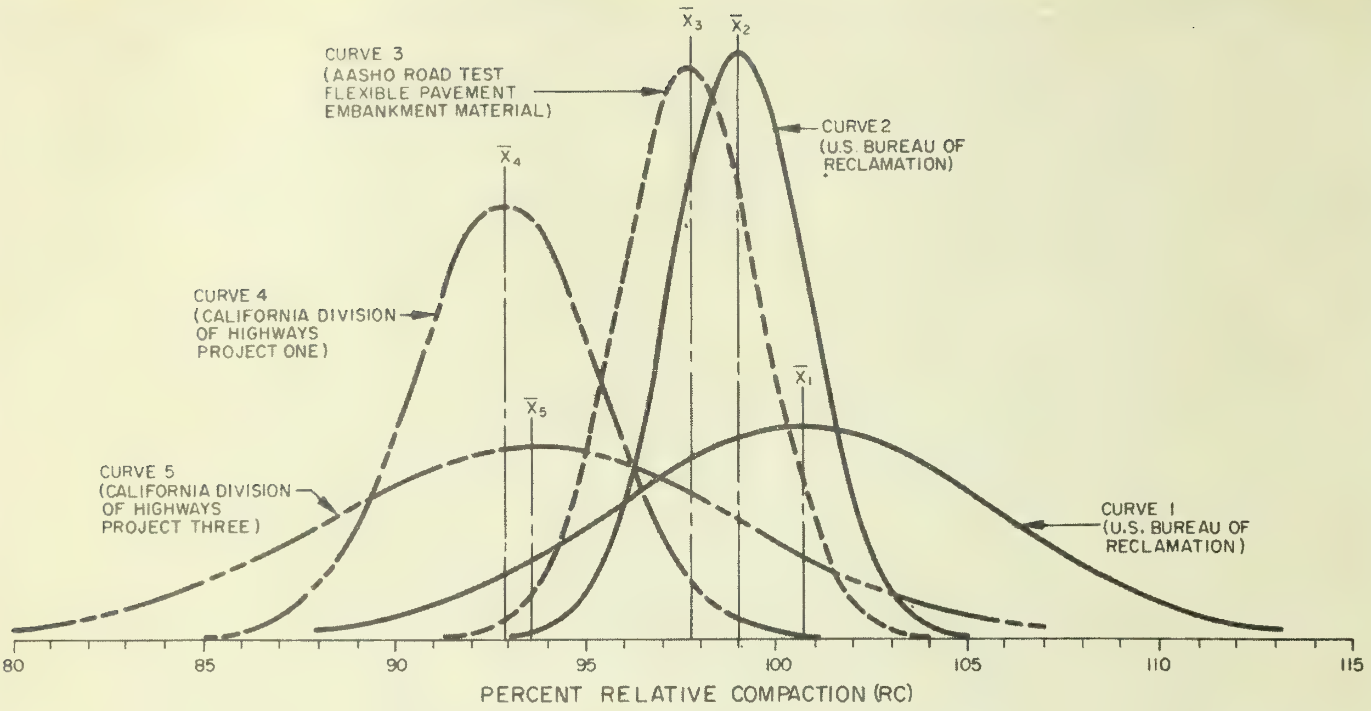
Current practices

It has long been the custom to define desirable compaction as the degree of compaction that is above some lower limit set by engineering judgment and based on experience with various materials and performance requirements. This lower limit is described as percent of a maximum density determined in the laboratory for each type of soil to be encountered on a project.

Although most engineers have recognized that measurements of density are not absolutely reproducible in themselves, and that material variations in any embankment or base may be the rule rather than the exception, the extent of the density-measurement variations seldom has been determined. Because these variations have not been recognized, misunderstanding exists within the engineering profession and between engineers and nontechnical people. Engineers, as well as public agencies, have often been criticized when it was shown in subsequent test results that accepted embankments and base courses failed to meet minimum requirements even though no evidence of unsatisfactory performance existed.

A look at present specifications and compaction-measurement methods emphasizes the misunderstanding that exists. To develop measurement criteria, a series of laboratory compaction tests is run to establish the maximum density and optimum moisture content for each soil or base type. It is common practice to run one series of standard compaction tests for each material although it is fairly common knowledge that if a second series was run on another portion of the same material the results of the two tests might differ by several pounds per cubic foot. Frequently, the field technician uses density values established in the laboratory to determine percent compaction at the construction site by comparing the results of field tests with the laboratory-developed curves. He must make a judgment as to whether the type soil he has tested is the same as that for which a curve has been established. It is often apparent that his decision on which curve to use is based on density comparisons rather than on soil type comparisons. Present day construction methods further contribute to the difficulties of the technician, who seldom will encounter material in the field that is an exact duplicate of the material tested in the laboratory. Excavation and spreading of large quantities of materials nearly always result in mixtures of types or variations of type from spot to spot in the fill.

Not only are the methods of applying the test results difficult to rationalize, but the tests themselves are not reproducible to the extent necessary for exact measurement. Several years were futilely spent in comparing



CURVE NO.	1	2	3	4	5
MINIMUM SPECIFICATION LIMIT, % RC	98	98	95	90	90
COMPACTION TEST METHOD	PROCTOR E-11	PROCTOR E-11	AASHO T-99	CALIF. 216	CALIF. 216
AVERAGE COMPACTION	100.7	99.0	97.7	92.9	93.6
STANDARD DEVIATION	5.0	1.8	1.9	2.4	5.5
APPROXIMATE % LESS THAN MINIMUM SPECIFICATION LIMIT	29.5	28.9	7.8	11.3	25.6

Figure 2.—Normal distribution curves from three organizations.

Table 1.—Percent relative compaction for different test methods

Compacted components	Sand cone			Portable nuclear			Roadlogger		
	Mean	[1] σ_{st}^2	[2] σ_o	Mean	[1] σ_{st}^2	[2] σ_o	Mean	[1] σ_{st}^2	[2] σ_o
Embankments	99.1	3.46	4.46	99.0	2.25	4.55	100.2	2.17	4.44
Bases and subbases	98.7	2.16	2.92	98.1	1.32	2.89	98.0	0.60	2.48

[1] σ_{st}^2 Sampling and testing variance.

[2] σ_o Overall standard deviation.

the results of nuclear measurements to those of conventional measurements. Only recently has it been demonstrated that the nuclear device is capable of producing more precise measurements than can be obtained by conventional methods.

One major factor that influences the variation in conventional-density test results is the common practice of removing the larger particles, greater than $^3/_4$ inch, from the samples tested in the laboratory. The effect of these larger particles on field results is estimated by empirical mathematical equations and superimposed on the results of the laboratory tests. Many laboratories realize the fallacy of this practice and are using larger molds in their tests.

Sampling

Selective sampling by the inspector, often as ordered by the engineer, has played an important part in the failure to recognize the magnitude of the actual variations occurring in embankment and base construction. When the inspector has the opportunity to select the test site, he has three alternatives: (1) To select an average condition, (2) to select the poorest condition, and (3) to select the best condition. The general custom in the State or the specific practice of the engineer on the job may well determine the site he selects for test. Regardless of his choice, the results of his tests will reflect only the condition he is selecting and not the variability of results or the true overall level of compaction.

Valid measurements of the actual quality of the compaction can be made only if the sample is a true representation of the total compacted material. It is possible to obtain a representation of the entire mass only when the sampling program is so designed that each element in the mass has an equal chance of being one of the elements of the sample. Of course, the greater the number of elements sampled, the better will be the representation.

The Statistical Approach

Although many questions concerning the required level of compaction and the methods of obtaining it are still unanswered, almost everyone agrees that uniformity of support is the principal requirement of good embankment and base-course construction. As a result of recent measurements obtained in research, the need for a change in methods of control has become apparent. Any such change must be directed toward controlling uniformity as well as degree of compaction.

The use of statistical concepts to establish the requirements of specifications and to aid in the analysis of test data provides much of the needed improvement. The specification either designates a target percent-compaction value and the allowable variations about this value or designates a lower limit to be met by a given percentage of the construction, when a valid statistical analysis of test results is performed.

A statistically based specification requires that a contractor submit a *lot* of predetermined size to the buyer for acceptance. Each *lot* is

Table 2.—Average, range, and standard deviations of percent compaction of subgrade and subbase projects

Project	Average compaction	Range of compaction	Standard deviation
	Percent	*Percent*	*Percent*
S-1	100.6	84-116	5.3
S-2	96.8	80-110	5.7
S-3	98.2	84-108	4.5
B-1	89.4	82-98	3.3
B 2	91.7	84-100	3.1
B-3	93.6	86-100	2.3

Table 3.—Average differences between sand-cone density tests for replicate tests

Project	Replicates	Average difference between sand-cone density values for replicate tests
	Number	*Lb. per cu. ft.*
S-1	48	3.32
S-2	48	4.95
S-3	49	4.18
B-1	51	4.15
B-2	55	3.35
B-3	50	2.24

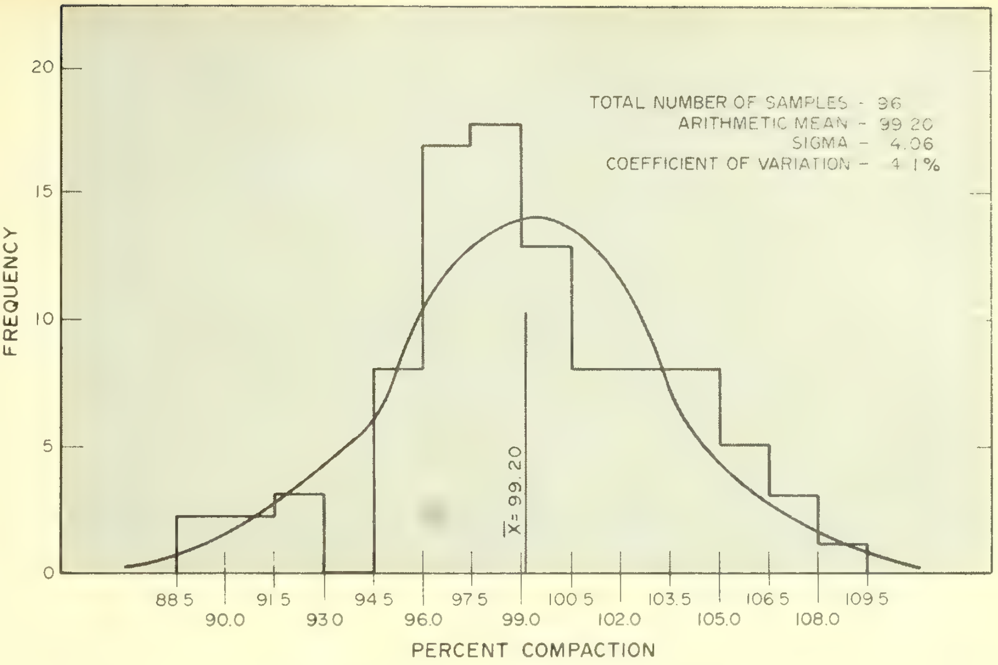


Figure 3.—Soil aggregate base, percent compaction.

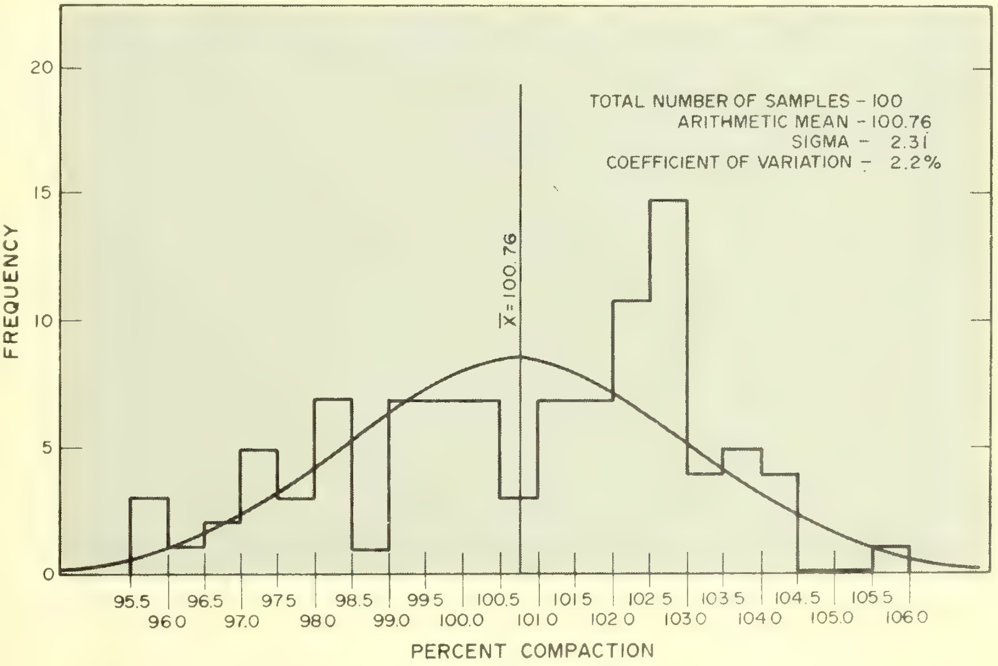


Figure 4.—Selected soil subbase, Class 4, percent compaction.

Table 4.—Maximum dry density and optimum moisture content values for duplicate field samples for Project S-1

Test number	Maximum dry density	Optimum moisture content	Liquid limit	Plasticity index	AASHO classification
	Lb. per cu. ft.	*Percent*			
27 A	118.7	13.7	32.5	13.5	[illegible]
27 B	119.3	13.5	36.0	15.2	[illegible]
32 A	117.4	14.4	33.9	14.3	[illegible]
32 B	115.8	15.4	34.6	13.9	[illegible]
34 A	113.5	15.4	41.5	18.4	[illegible]
34 B	110.6	14.3	41.6	16.9	[illegible]
36 A	123.0	13.4	30.2	12.3	[illegible]
36 B	119.7	13.3	31.0	11.9	[illegible]
37 A	121.2	12.6	28.1	8.6	A-4
37 B	122.6	11.0			
38 A	112.6	17.3	41.3	17.6	A-7-6 (11)
38 B	113.5	15.9	37.6	14.6	A-6 (9)

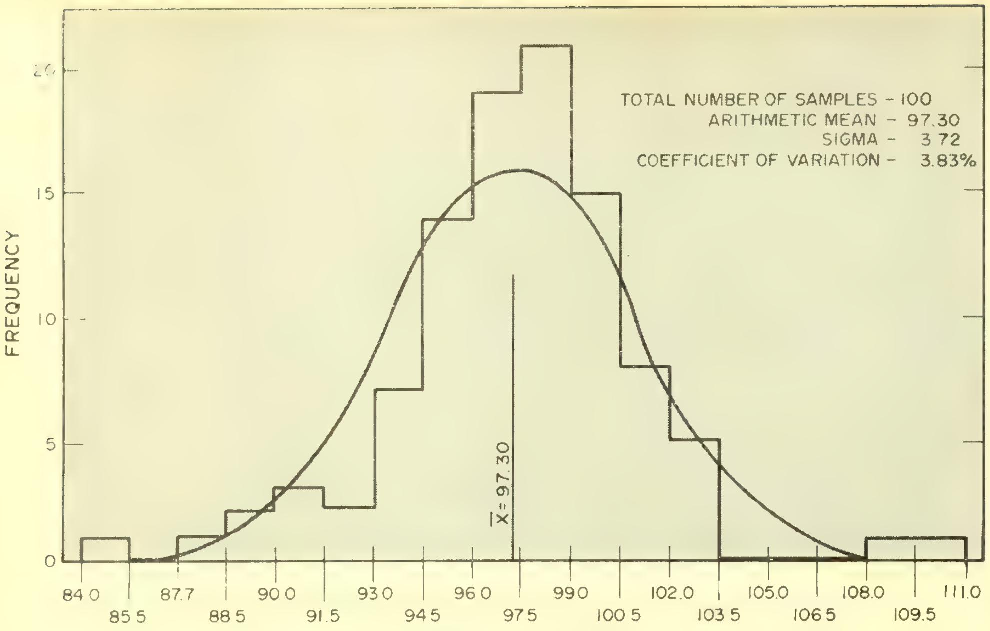


Figure 5.—Embankment, percent compaction.

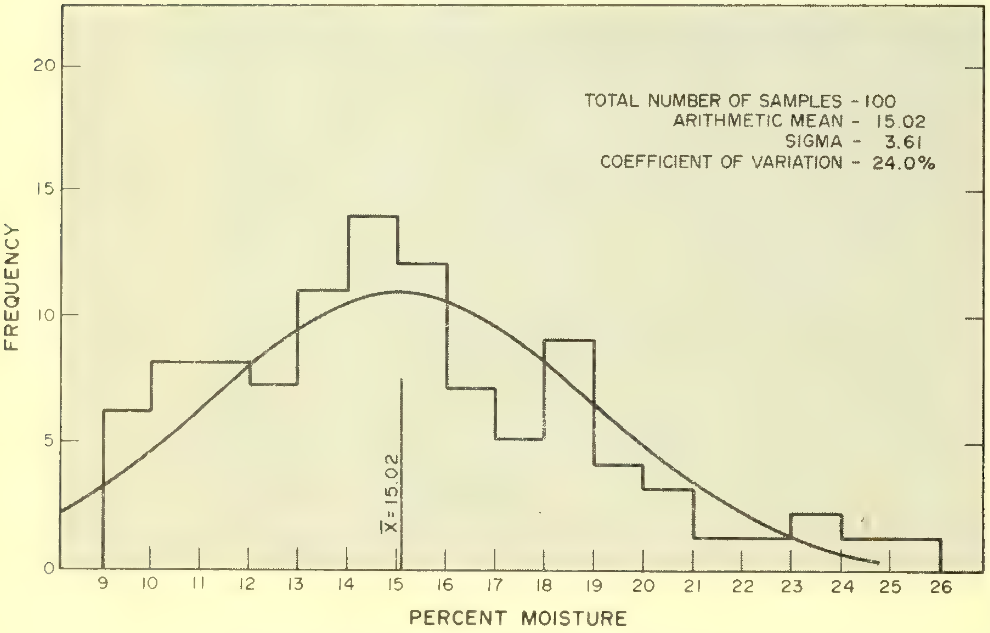


Figure 6.—Embankment, percent moisture.

evaluated on the basis of the results of a specified sampling and testing program. This program entails the performance of a specified number of standard tests at random locations [illegible] lot submitted. The data analysis procedures to establish compliance and the [illegible] be taken if noncompliance is indicated are also spelled out.

Several States have developed specifications for embankment or base construction that are great improvements over present methods and are based partly on statistics, even though they are not strictly in accordance with concepts recommended in this series of discussions.

Virginia, for example, is using a control strip technique for control of the compaction of aggregate base. The following special provisions were extracted from a paper (2), presented at the 46th Annual Meeting of the Highway Research Board:

"Virginia Department of Highways Special Provisions For Nuclear Field Density Testing of Aggregate Base and Surface Courses

"Section 308 of the 1966 edition of the Road and Bridge Specifications is amended in this contract to require the construction of density control strips for the purpose of using the nuclear field density testing device. The revisions are as follows:

"At the beginning of the work the Contractor shall build a control strip of the material on an approved and stable subgrade for the purpose of the Engineer's determining density requirements for the project. This control strip will be at least 400 square yards in area and of the same material and depth to be used in the remainder of the work. Compaction will be carried out with conventional rollers approved by the Engineer until no appreciable increase in density is accomplished or until in the opinion of the Engineer no appreciable increase in density will be obtained by additional rolling. Upon completion of the rolling, the density of the strip will be determined by use of a portable nuclear test device.

"The compaction of the remainder of the aggregate base course material shall be governed by the density of the control strip. The material shall be tested by sections of approximately 2,800 square yards each. The mean density of 5 randomly selected sites from the test section shall be at least 98 percent of the mean density of 10 tests taken from the approved control strip. Placing, compacting and individual testing may be done in subsections of approximately 280 square yards each. When the mean of the test section is less than 98 percent of the control strip mean the Contractor may be required to rework the entire section. Also, each individual test value shall be at least 95 percent of the mean value of the control strip. When an individual test value is less than 95 percent of the control strip mean, the contractor shall be required to rework the area represented by that test.

"Each test section shall be tested for thickness and any deficiency outside the allowable tolerance shall be corrected by scarifying, placing additional material, remixing, reshaping and recompacting to the specified density.

"A new control strip may be requested when:

(1) A change in the source of the material is made, or
(2) a change in the material from the same source is observed, or
(3) ten (10) test sections have been approved without the construction of additional control strips.

"Note: The Contractors' attention is directed to the fact that the method for determining density and the requirements for density as described in Section 308.05 have been replaced by the method of determination and requirements for density stated hereinabove."

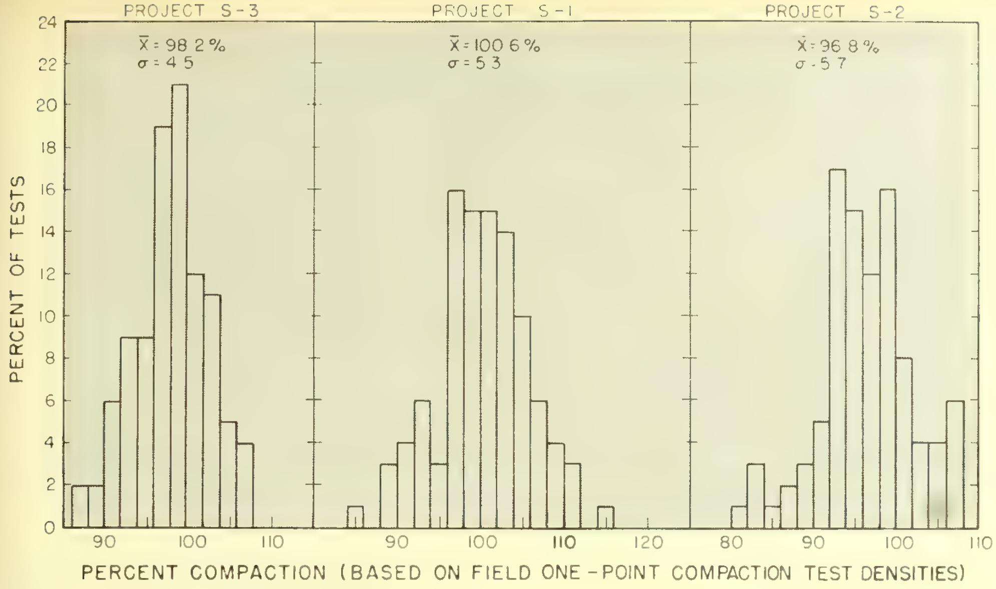


Figure 7.—Frequency histograms—percent compaction of subgrade materials for three projects.

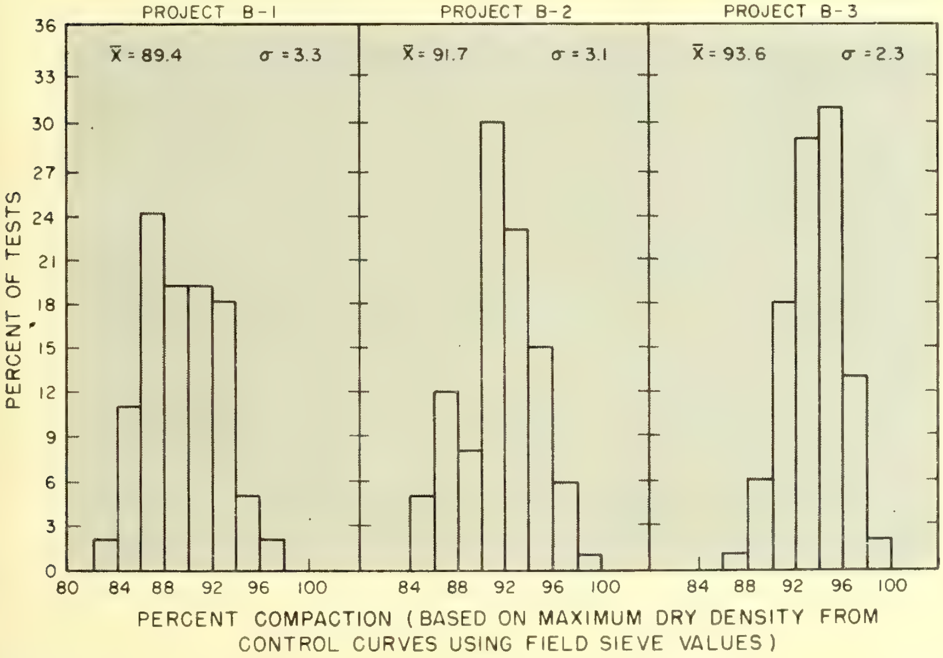


Figure 8.—Frequency histograms—percent compaction of subbase materials for three projects.

Reliability of nuclear testing

The Virginia specification is an excellent example of the more rapid methods that can be used to allow the testing of a more representative sample of completed work. The increased number of test results available for making a decision assures a higher confidence in the decision. The following advantages are claimed by the Virginia Department of Highways:

- Nuclear tests can be made quickly and easily.

- A field control strip provides a practical achievable density.

- The speed of nuclear testing permits determination to be made for each section of material. This procedure provides a sound statistical basis for decisionmaking.

The reliability of nuclear-gage test results is substantiated by tests made in a number of States. For example, the data in table 1 are from two studies in Utah (*3*, *4*). The absolute values of the standard deviations presented in the table have little significance with respect to testing variability because much of the indicated variation is probably caused by actual density variations. However, it is significant that the sampling and testing variance is smaller and that there is no significant difference in the means. The results of the nuclear tests are as good, if not better, than those of the conventional tests; consequently, it can be stated safely that the testing error of nuclear methods is probably no greater than that of conventional methods.

Reported variations in compaction

The variation in density of accepted embankments and bases has been found much greater than had been expected when the Public Roads research program (see part 1 of this article, Feb. 1969 issue) was initiated. Because of this variability, compliance with specifications, as computed by statistical methods, is lower than had been expected. Therefore, designers must judge whether present construction is sufficient for their purposes. If present construction is satisfactory, then specifications should be changed to allow for the existing variation. If better construction is needed, then it is important that specifications and methods be changed to assure better uniformity in embankments and base courses.

Research is showing that overall standard deviation, a measure of variability, is not in itself a true indication of contractor-performance variability. A good contractor may take the same care in constructing two embankments but the variability of test results may be much greater in one than in the other. If the composition of the material itself is more variable, then the results of the compaction process are also going to be more variable.

The variation of density in embankments, with respect to material and process changes, is shown in figures 1 and 2. Figure 1, extracted from a California report (*5*), presents the distributions of the results of density tests on three projects. Project No. 1 was constructed with homogeneous, fine grained soils; Project No. 3 with an extremely heterogeneous soil; and Project No. 2 with a soil of intermediate variability, with respect to the other two. The specification on each of the projects stipulated that the material be compacted to no less than 90 percent relative compaction. It has been shown by many of the research test results obtained after acceptance by normal control procedures, that the construction does not meet specification requirements when the data are analyzed on a statistical basis and the total material is considered.

Figure 2, also from the California report, is presented to show that variability of compaction test results is not unique to the highway industry.

Figures 3 and 4 have been extracted from an Alabama Research Report (*6*) to show indicated variation in density of compacted base and subbase materials. The standard deviations of 1.06 and 2.31 percent are in line with values reported by other States. Figures 5 and 6 are from the same report; the data reported

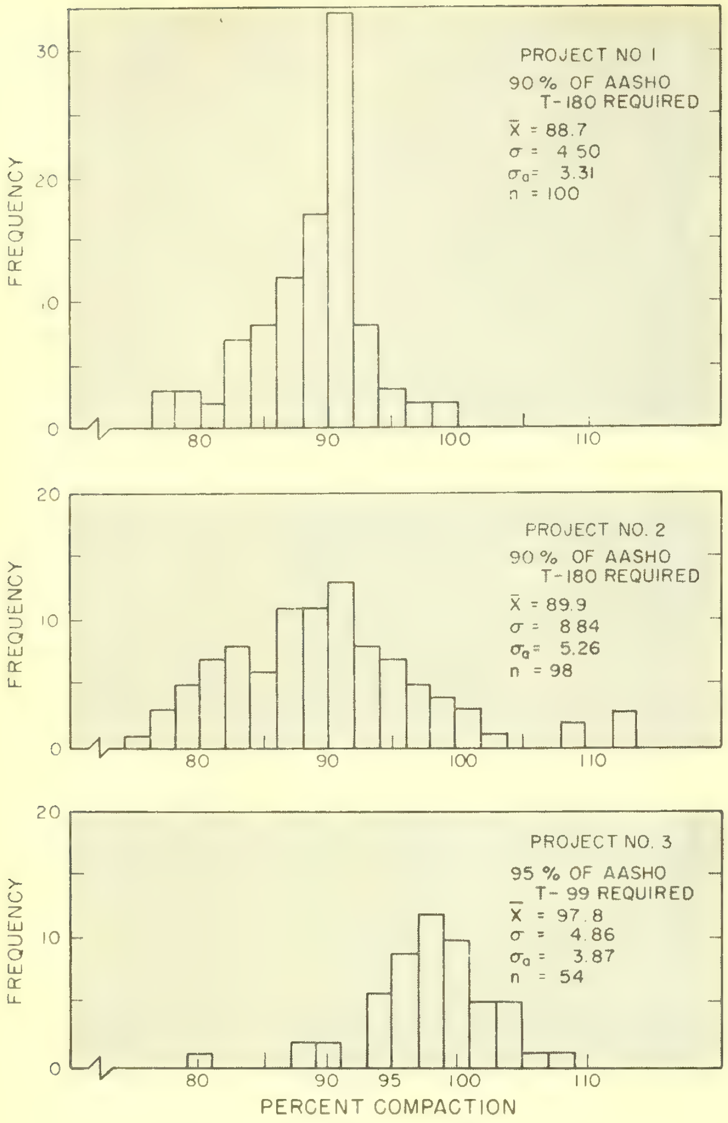


Figure 9.—Percent compaction—research data.

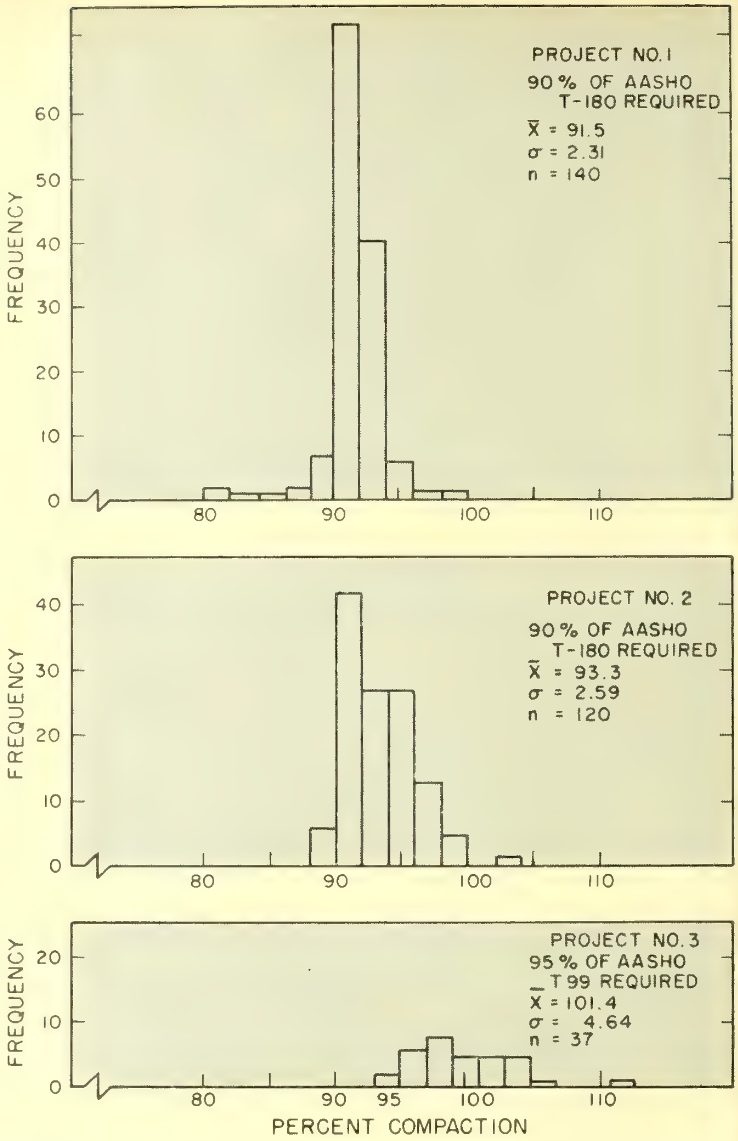


Figure 10.—Percent compaction—highway-department data.

by California for compacted densities of embankment materials are corroborated by the data in figure 5, and variations in moisture content are shown in figure 6. The large variation in moisture content is probably a major cause of the large variation in density.

From a research study performed by Purdue University for the State of Indiana in March 1967 (7), information concerning average density, range, and standard deviation are shown in table 2 for three subgrade and three subbase projects. The data for the study were obtained after the projects had been accepted under normal acceptance procedures. The specifications for the projects required a minimum density of 100 percent of standard laboratory maximum density.

The wide range of [illegible] standard deviations reported [illegible] part, due to variability contributed by test methods. The differences between replicate sand [illegible] density tests on the study projects [illegible] in table 3. The entire difference cannot be attributed to test error as there may be actual differences in the materials or densities even when the tests are taken side by side as was done in this study. However, the results show the magnitude of differences when an effort was made to eliminate material differences within the limitations of practical construction conditions.

Another contributing factor to the variation of test results is the difference in results of laboratory maximum-density and optimum-moisture-content tests. These differences for duplicate samples from Project S–1 of the Purdue University study are shown in table 4.

The data obtained in the Purdue University study are shown in figures 7 and 8 which are histograms of the percent compaction for the six projects.

Figures 9 and 10 were extracted from a report of a study conducted by the Engineering Experiment Station of North Dakota State University for the North Dakota State Highway Department (8). In figure 9 is shown the variability of compaction in three embankment projects previously accepted under current control and acceptance procedures. The distribution of test results on random samples are presented in the histograms in the figure. The mean $\overline{X}$; the overall standard deviation, σ; and the sampling and testing standard deviation, σ_α, of the distributions are tabulated. These standard deviations must be changed to variances in order to obtain the relationship between the material and the sampling and testing variability, $\sigma^2 - \sigma^2_\alpha = \sigma^2_m$. In figure 10, the information obtained during routine control and acceptance testing on the same three projects is presented. Comparison of the results presented in figures 9 and 10 emphasizes the advantages of random sampling in determining the true as-built conditions of any construction project and compliance with specifications.

Density-test results obtained with two types of nuclear gages and two different test methods on Project No. 1 of the North

Dakota study are shown in figure 11. These data substantiate the results of the Utah report in that the sampling and testing errors for the nuclear devices are smaller than those for the water-balloon method (fig. 9). It is of interest that the air-gap method indicates a higher average density than the water-balloon or contact-nuclear method. A similar nuclear study was performed on Project 2 with parallel results. In these tests, the manufacturer's calibration curves for the nuclear devices were verified before use.

Variations in Material Properties

Tables 5 and 6 were extracted from a California report (*9*) to show the variation of test results other than those of density tests. The data are from six projects selected as typical of material used for untreated base and subbase by the California Division of Highways. Again the data were obtained from random samples taken after the materials had been accepted as complying with the specifications for normal sampling methods. These materials were largely in substantial compliance with the specifications; however, there was considerable variation in the test results of the material properties, which may account for some of the variations in density and supporting capacity exhibited by the compacted material. The study did not include the determination of density variation of the in-place material.

Conclusions

The primary conclusion that can be drawn from the data presented in this discussion is that test results on base and embankment materials exhibit significant variation. These variations can be attributed to material variance, sampling variance, and testing variance. Many materials may be classified out-of-specification because of sampling and testing errors rather than failure of the material or construction to actually conform to specified requirements.

It should be apparent that improvement of sampling and testing methods must be a priority research and development item if field measurements on samples are to be used to accept construction materials and structures. More tests results must be used in the decision process to increase the validity of decisions. Rapid sampling and testing methods, together with random sampling and statistically valid decision plans, will alleviate many of the problems in current acceptance of construction.

The data and charts of this presentation clearly indicate a difference between the test results on random samples and those on representative samples. A true estimate of the actual quality of any material or construction can be obtained only when every item of the *lot* has a chance of being chosen as part of the sample. Sampling by choice cannot provide samples that will permit evaluation of both level and variability of material or construction. Randomizing sampling locations is a simple matter and should cause no serious problems for the inspector, especially when rapid nondestructive test methods, such as the nuclear gage, are available. State highway departments should take immediate steps to implement random sampling in the control and acceptance of base and embankment construction.

The data reported here concerning the variations in base and embankment construction should not be taken as an indictment of present construction. Although there is adequate information to indicate that improvement is needed in the testing and analysis of data, there is no specific information available to indicate that construction being accepted

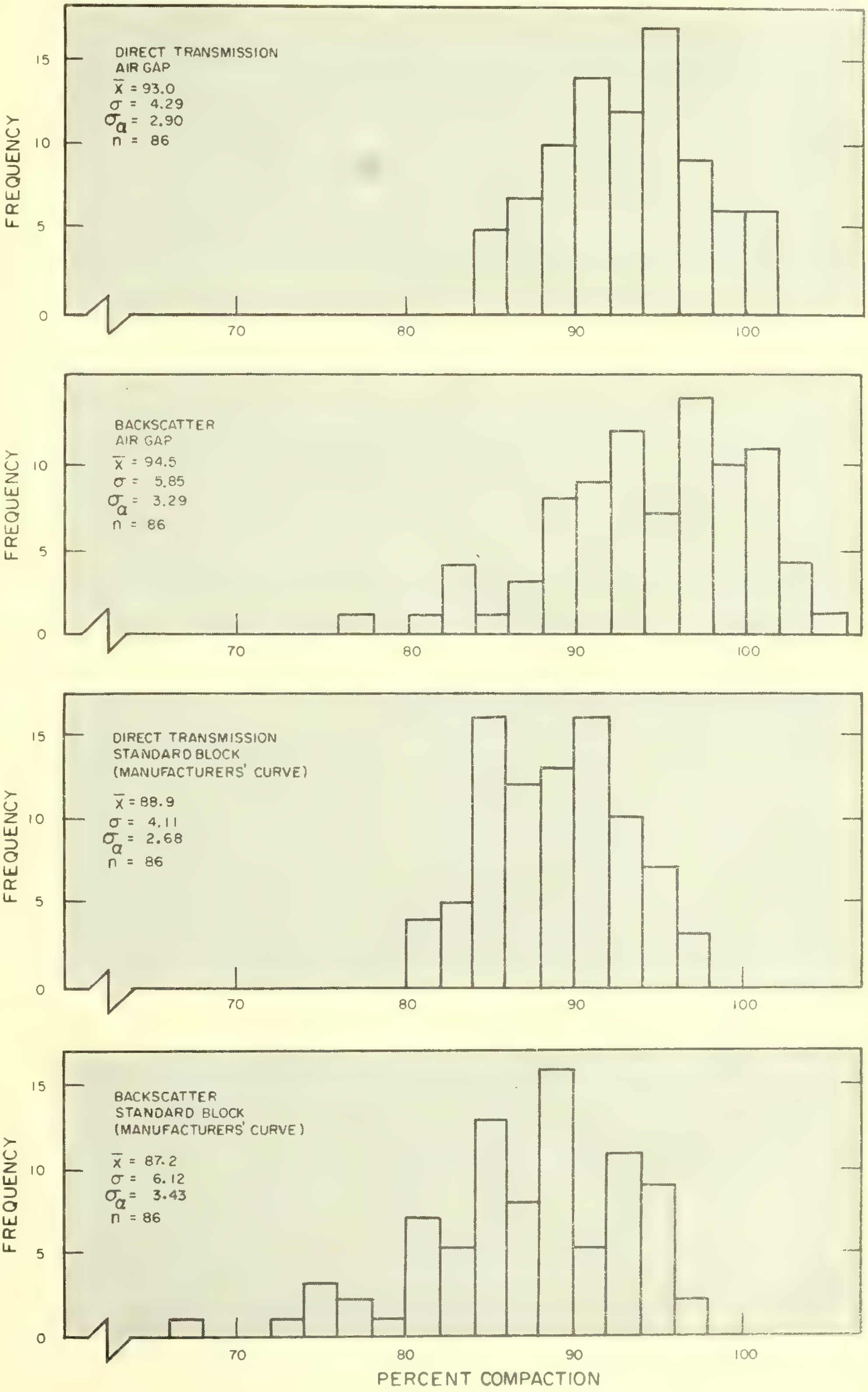


Figure 11.—Percent compaction—nuclear-instrument data.

under present procedures is not performing to design expectations. However, if economic considerations do not allow an intensive effort to reduce sampling and testing variation, as well as actual variation in density and moisture content, it is imperative that recognition be given the variations occurring in present construction and that current specifications be revised accordingly.

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(*9*) *A Statistical Analysis of Untreated Base and Subbase Materials,* State of California, Department of Public Works, Division of Highways, March 1967.

Table 5.—Summary of test results, untreated aggregate base

Test	[1] n	[2] $\overline{X}$	[3] σ	Range of results	Specification requirement	Amount not complying with specification
PROJECT B–1						
						Percent
R value	200	81.9	1.3	78–85	[4] 78	0
Sand equivalent	200	42.9	4.0	33–58	[4] 30	0
Percent passing #4 sieve	200	50.9	3.1	35–55	35–55	0
Percent passing #30 sieve	200	23.8	2.5	15–30	10–30	0
Percent passing #200 sieve	200	6.0	0.7	4–8	3–9	0
PROJECT B–2						
R value	200	79.9	2.4	72–85	[4] 75	2.5
Sand equivalent	200	30.6	6.1	24–63	[4] 30	56
Percent passing #4 sieve	200	58.1	2.8	51–67	35–65	2.5
Percent passing #30 sieve	200	27.3	2.3	22–36	([5])	---
Percent passing #200 sieve	200	7.9	1.1	4–10	3–12	0
PROJECT B–3						
R value	200	79.7	1.5	78–83	[4] 78	0
Sand equivalent	200	59.2	4.0	48–68	[4] 30	0
Percent passing #4 sieve	200	52.7	5.7	40–71	35–55	33
Percent passing #30 sieve	200	23.4	2.9	15–31	10–30	0.5
Percent passing #200 sieve	200	4.6	0.95	3–12	3–9	0.5

[1] n=Number of samples.
[2] $\overline{X}$=Arithmetic mean.
[3] σ=Standard deviation.
[4] Minimum.
[5] None.

Table 6.—Summary of test results, untreated aggregate subbase

Test	[1] n	[2] X	[3] σ	Range of results	Specification requirement	Amount not complying with specification
PROJECT S–1						
						Percent
R value	200	68.8	6.4	47–80	[4] 60	9.5
Sand equivalent	200	30.2	4.0	22–46	[4] 25	5.0
Percent passing #4 sieve	200	49.5	4.3	40–60	35–65	0
Percent passing #200 sieve	200	7.8	1.3	5–13	3–11	0.5
PROJECT S–2						
R value	188	77.2	3.1	66–83	[4] 60	0
Sand equivalent	188	36.2	8.5	14–69	[4] 25	2.0
Percent passing #4 sieve	188	72.6	6.5	60–91	30–100	0
Percent passing #200 sieve	188	10.0	1.8	5–16	0–20	0
PROJECT S–3						
R value	200	70.9	8.9	42–84	[4] 55	6.5
Sand equivalent	200	29.2	2.7	21–37	[4] 25	4.5
Percent passing #4 sieve	200	45.0	6.6	23–57	35–80	4.5
Percent passing #200 sieve	200	8.6	1.7	4–12	5–35	1.0

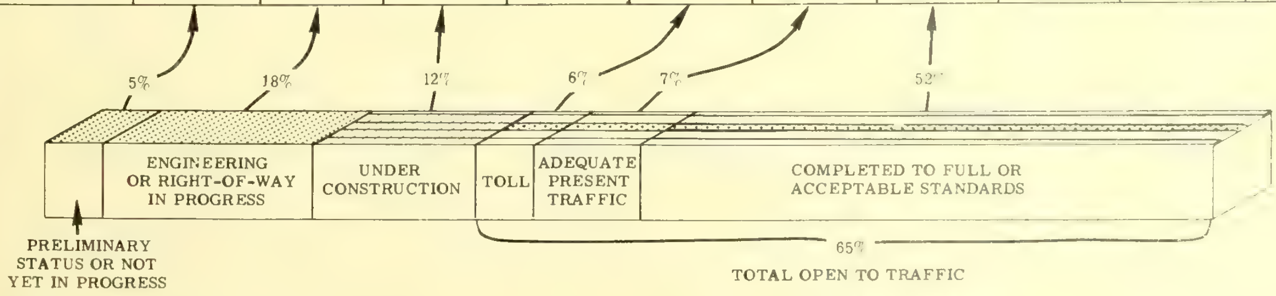
[1] n=Number of samples.
[2] X=Arithmetic mean.
[3] σ=Standard deviation.
[4] Minimum.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

IMPROVEMENT STATUS OF SYSTEM MILEAGE AS OF DECEMBER 31, 1968

TABLE 1

STATE	PRELIMINARY STATUS OR NOT YET IN PROGRESS 1/	WORK IN PROGRESS			OPEN TO TRAFFIC				TOTAL DESIGNATED SYSTEM MILEAGE	STATE
		ENGINEERING OR RIGHT-OF-WAY	UNDER CONSTRUCTION	TOTAL UNDERWAY	TOLL FACILITIES	IMPROVED TO STANDARDS ADEQUATE FOR PRESENT TRAFFIC	COMPLETED TO FULL OR ACCEPTABLE STANDARDS	TOTAL OPEN TO TRAFFIC		
ALABAMA	19.2	211.2	181.4	392.6	-	141.1	343.7	484.8	896.6	ALABAMA
ARIZONA	5.9	162.2	218.3	380.5	-	240.8	545.0	785.8	1,172.2	ARIZONA
ARKANSAS	-	41.9	109.7	151.6	-	4.3	363.0	367.3	518.9	ARKANSAS
CALIFORNIA	108.8	412.8	304.2	717.0	10.2	304.4	1,133.3	1,447.9	2,273.7 2/	CALIFORNIA
COLORADO	158.6	112.2	60.4	172.6	-	112.8	531.9	644.7	975.9	COLORADO
CONNECTICUT	51.6	23.1	11.2	34.3	16.4	47.3	197.5	261.2	347.1	CONNECTICUT
DELAWARE	-	9.4	2.1	11.5	14.3	0.9	13.9	29.1	40.6	DELAWARE
FLORIDA	271.2	304.2	111.7	415.9	44.8	-	681.6	726.4	1,413.5	FLORIDA
GEORGIA	38.8	295.0	162.9	457.9	-	6.9	643.6	650.5	1,147.2	GEORGIA
HAWAII	11.6	22.4	5.7	28.1	-	1.6	10.5	12.1	51.8	HAWAII
IDAHO	-	133.7	80.8	214.5	-	96.3	300.8	397.1	611.6	IDAHO
ILLINOIS	118.8	291.9	232.2	524.1	155.7	143.0	780.8	1,079.5	1,722.4	ILLINOIS
INDIANA	14.0	197.6	141.8	339.4	156.9	15.4	603.4	775.7	1,129.1	INDIANA
IOWA	74.8	138.9	52.4	191.3	3.6	-	514.1	517.7	783.8	IOWA
KANSAS	19.6	80.5	70.1	150.6	185.9	0.3	464.1	650.3	820.5	KANSAS
KENTUCKY	-	153.4	107.9	261.3	39.2	4.2	433.9	477.3	738.6	KENTUCKY
LOUISIANA	30.0	186.3	179.6	365.9	-	6.4	301.0	307.4	703.3	LOUISIANA
MAINE	1.7	32.7	1.9	34.6	58.0	99.4	118.3	275.7	312.0	MAINE
MARYLAND	25.2	7.2	30.5	37.7	53.0	70.9	173.3	297.2	360.1	MARYLAND
MASSACHUSETTS	19.0	31.2	31.3	62.5	134.4	27.4	223.7	385.5	467.0	MASSACHUSETTS
MICHIGAN	92.6	165.0	25.9	190.9	4.8	44.4	841.1	890.3	1,173.8	MICHIGAN
MINNESOTA	9.4	240.4	210.8	451.2	-	30.3	422.5	452.8	913.4	MINNESOTA
MISSISSIPPI	-	125.6	85.4	211.0	-	19.2	448.1	467.3	678.3	MISSISSIPPI
MISSOURI	26.6	258.6	34.2	292.8	0.3	160.8	665.4	826.5	1,145.9	MISSOURI
MONTANA	24.6	465.3	101.8	567.1	-	301.8	292.5	594.3	1,186.0	MONTANA
NEBRASKA	1.9	72.8	31.8	104.6	0.2	13.6	359.2	373.0	479.5	NEBRASKA
NEVADA	-	128.7	32.5	161.2	-	5.3	368.1	373.4	534.6	NEVADA
NEW HAMPSHIRE	11.3	25.3	7.6	32.9	22.0	14.8	134.1	170.9	215.1	NEW HAMPSHIRE
NEW JERSEY	53.0	87.7	58.3	146.0	46.3	26.4	113.5	186.2	385.2 3/	NEW JERSEY
NEW MEXICO	37.5	185.5	91.7	277.2	-	61.1	622.6	683.7	998.4	NEW MEXICO
NEW YORK	152.4	44.0	81.2	125.2	491.8	53.3	532.5	1,077.6	1,355.2	NEW YORK
NORTH CAROLINA	67.2	195.4	108.3	303.7	-	17.0	449.3	466.3	837.2	NORTH CAROLINA
NORTH DAKOTA	62.6	38.8	77.2	116.0	-	51.9	340.3	392.2	570.8	NORTH DAKOTA
OHIO	12.3	162.1	226.9	389.0	206.4	55.0	871.4	1,132.8	1,534.1	OHIO
OKLAHOMA	9.3	53.9	144.4	198.3	174.1	23.3	401.7	599.1	806.7	OKLAHOMA
OREGON	19.2	65.5	2.5	68.0	-	111.2	537.7	648.9	736.1	OREGON
PENNSYLVANIA	39.1	115.9	275.6	391.5	360.2	8.3	781.6	1,150.1	1,580.7	PENNSYLVANIA
RHODE ISLAND	27.9	9.1	14.0	23.1	-	10.9	36.8	47.7	98.7	RHODE ISLAND
SOUTH CAROLINA	73.7	92.2	196.1	288.3	-	15.1	378.7	393.8	755.8	SOUTH CAROLINA
SOUTH DAKOTA	-	161.4	93.2	254.6	-	60.3	364.3	424.6	679.2	SOUTH DAKOTA
TENNESSEE	7.5	262.1	150.7	412.8	-	90.5	534.3	624.8	1,045.1	TENNESSEE
TEXAS	139.0	559.3	395.9	955.2	-	285.9	1,786.1	2,072.0	3,166.2	TEXAS
UTAH	50.8	374.0	208.0	582.0	-	22.6	277.7	300.3	933.1	UTAH
VERMONT	-	116.2	31.2	147.4	-	4.4	168.6	173.0	320.4	VERMONT
VIRGINIA	9.8	216.5	158.6	375.1	37.6	44.9	600.8	683.3	1,068.2	VIRGINIA
WASHINGTON	76.8	125.5	79.5	205.0	-	196.0	276.9	472.9	754.7	WASHINGTON
WEST VIRGINIA	29.5	158.6	54.4	213.0	87.2	0.3	184.7	272.2	514.7	WEST VIRGINIA
WISCONSIN	105.5	1.7	39.2	40.9	-	24.7	392.1	416.8	563.2	WISCONSIN
WYOMING	82.4	76.3	101.2	177.5	-	30.3	623.8	654.1	914.0	WYOMING
DISTRICT OF COLUMBIA	9.9	7.9	1.7	9.6	-	2.9	7.2	10.1	29.6	DISTRICT OF COLUMBIA
PENDING	40.2 4/								40.2 4/	PENDING
TOTAL	2,240.8	7,439.1	5,215.9	12,655.0	2,303.3	3,109.9	22,191.0	27,604.2	42,500.0	TOTAL



1/ Includes all routes and route segments added to the system under the 1,500 mile expansion authorized by the Federal-Aid Highway Act of 1968.

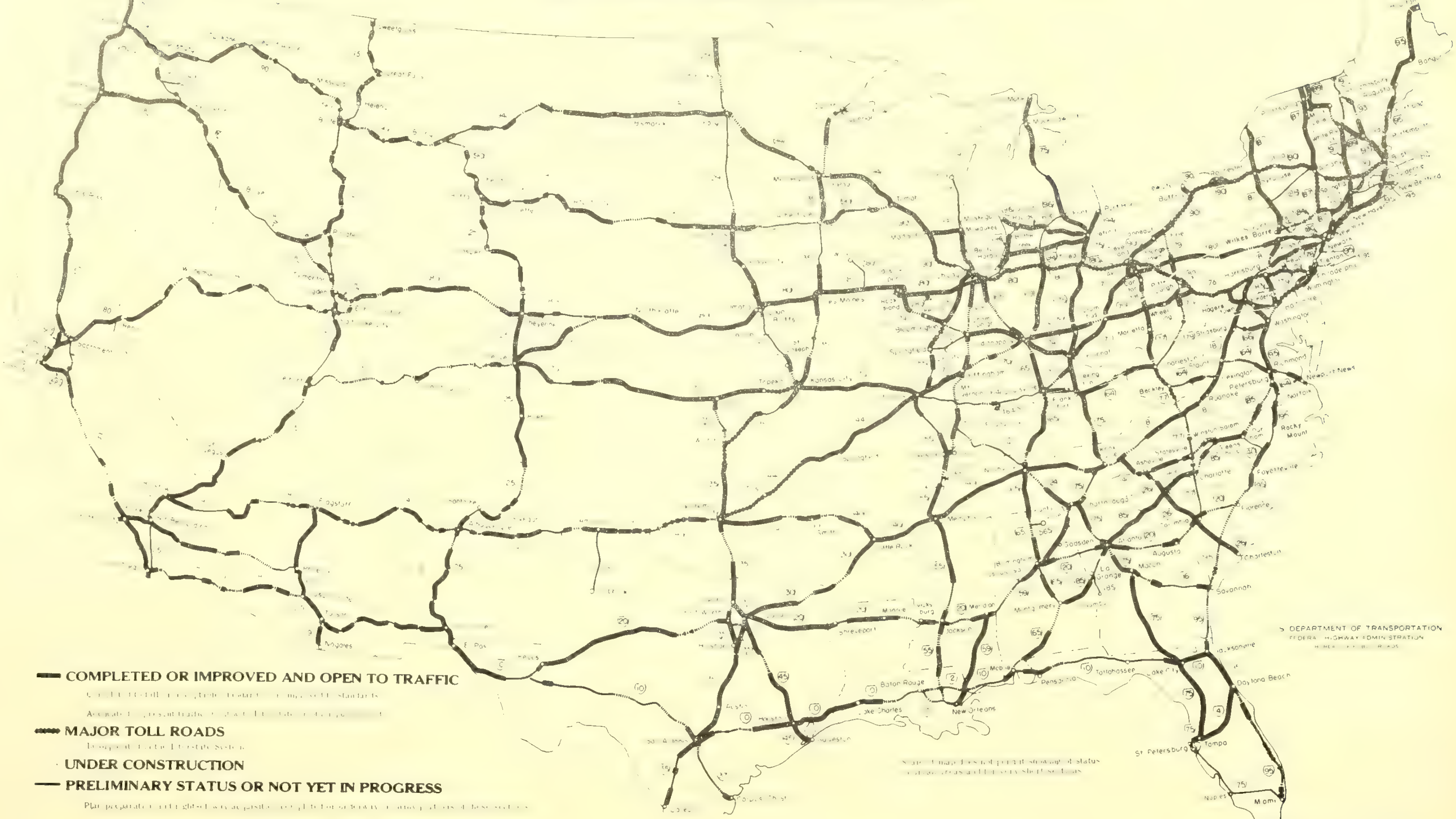
2/ Excludes the 17.2 mile Century Freeway (I-105) which was added to the system under the "Howard Bill."

3/ Excludes the 34.4 mile Trenton-Asbury Park Spur (I-195) which was added to the system under the "Howard Bill" but includes that portion of I-278 mileage (7.0) deleted under the same bill.

4/ Consists of mileage which has not been assigned to any specific route and is a reserve for final measurement of the system.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF DECEMBER 31, 1968



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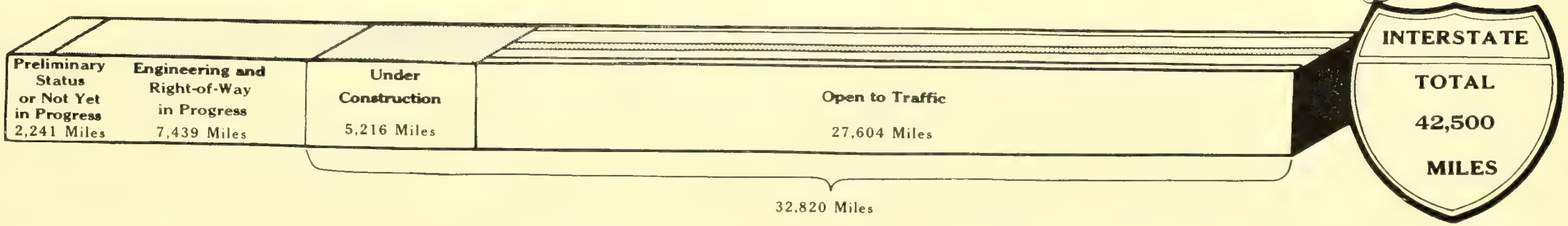

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June 1969

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U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

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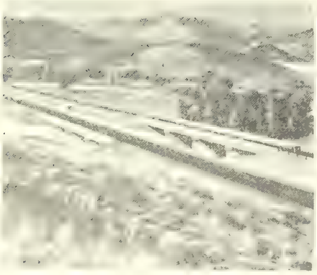

COVER

Twin bridge structures on a completed section of Interstate Highway 80 east of Reno, Nev.

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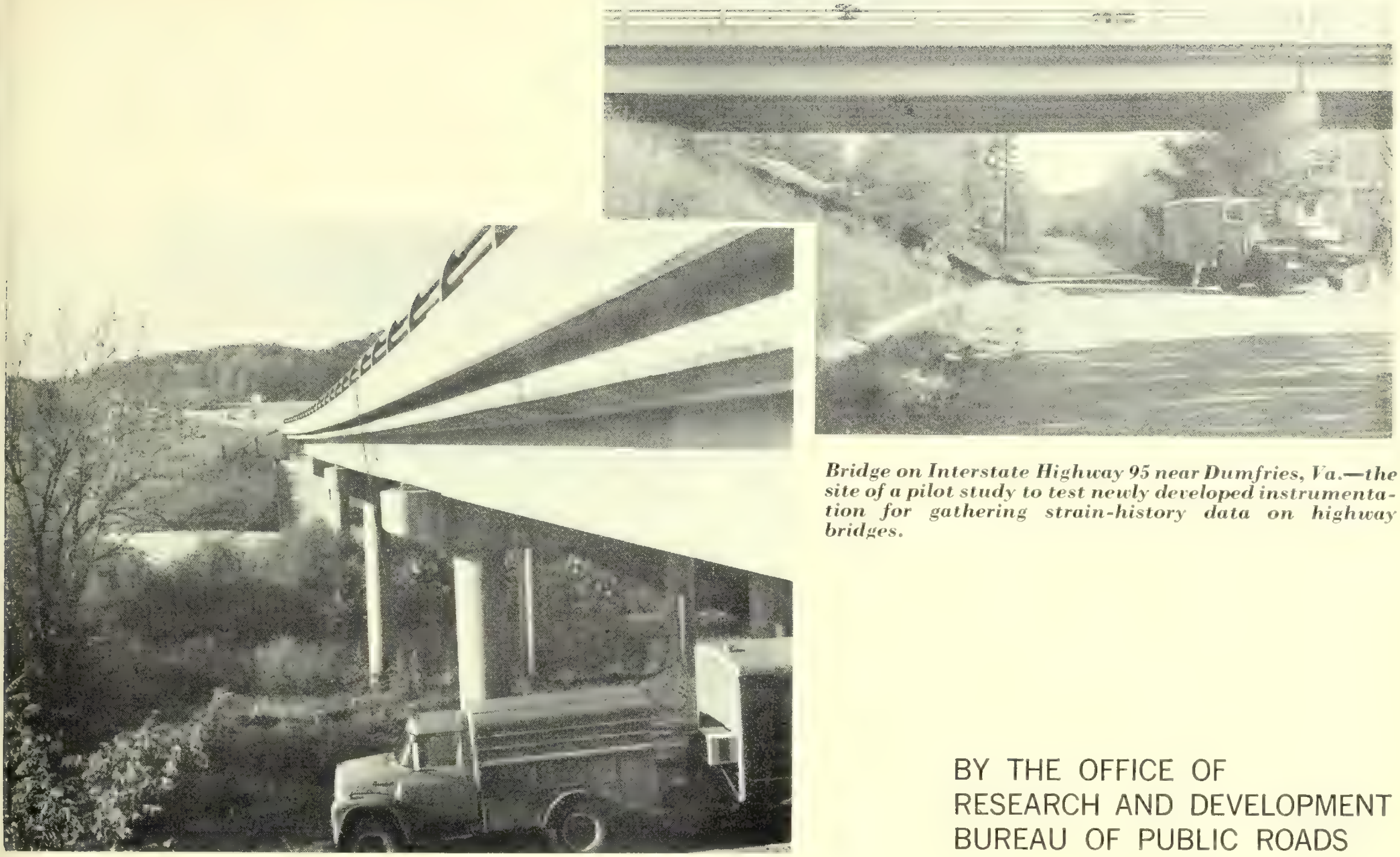

Bridge on Interstate Highway 95 near Dumfries, Va.—the site of a pilot study to test newly developed instrumentation for gathering strain-history data on highway bridges.

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Acquisition of Loading History Data on Highway Bridges

Reported by [1] CHARLES F. GALAMBOS and WILLIAM L. ARMSTRONG, Structural Research Engineers, Structures and Applied Mechanics Division

A data acquisition system to monitor and digitize strains produced in highway bridges by ordinary truck traffic is described in this article. The system and an associated computer program were used in a pilot study, on an Interstate Highway bridge near Dumfries, Va. Strains were monitored during several sampling periods, and trucks were weighed and classified at a nearby weighing station. The performance of the system was satisfactory. Generally low strains, usually less than half of the calculated live-load design values, were measured and elements of the Dumfries bridge seem to be in no danger of fatigue failure.

Introduction

MUCH of the conservative approach to the design of highway bridges is due to the unpredictability of the precise nature of the loadings to which these structures will be subjected in their lifetime, and also to the lack of knowledge about what the behavior of the materials will be under the loadings.

Highway bridges are subjected to a variety of forces, ranging from the constant dead loads of the structures themselves, through slowly changing forces caused by material creep and temperature differentials, to an almost infinite variety of live loadings caused by moving vehicles. It would be impossible to base bridge designs on any precise knowledge of future loads, and it follows naturally that present design methods are approximations. Most bridges are designed to carry a static load produced by a design truck, with certain empirical allowances for increased stresses owing to dynamic loads.

Design methods provide for the damaging effects of repetitive loads only in a rather crude fashion—a fact that is becoming of increasing concern to bridge engineers. There is an obvious need to determine the loading history of highway bridges so that the stresses produced by traffic can be predicted and a more rational, realistic estimate of future stresses made.

Attempts at monitoring the stresses caused by traffic traversing a bridge have been cumbersome and time consuming because of laborious data reduction processes. The statis-

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[1] Presented at the 4 th annual meeting of the Highway Research Board, Washington, D.C., January 1969.

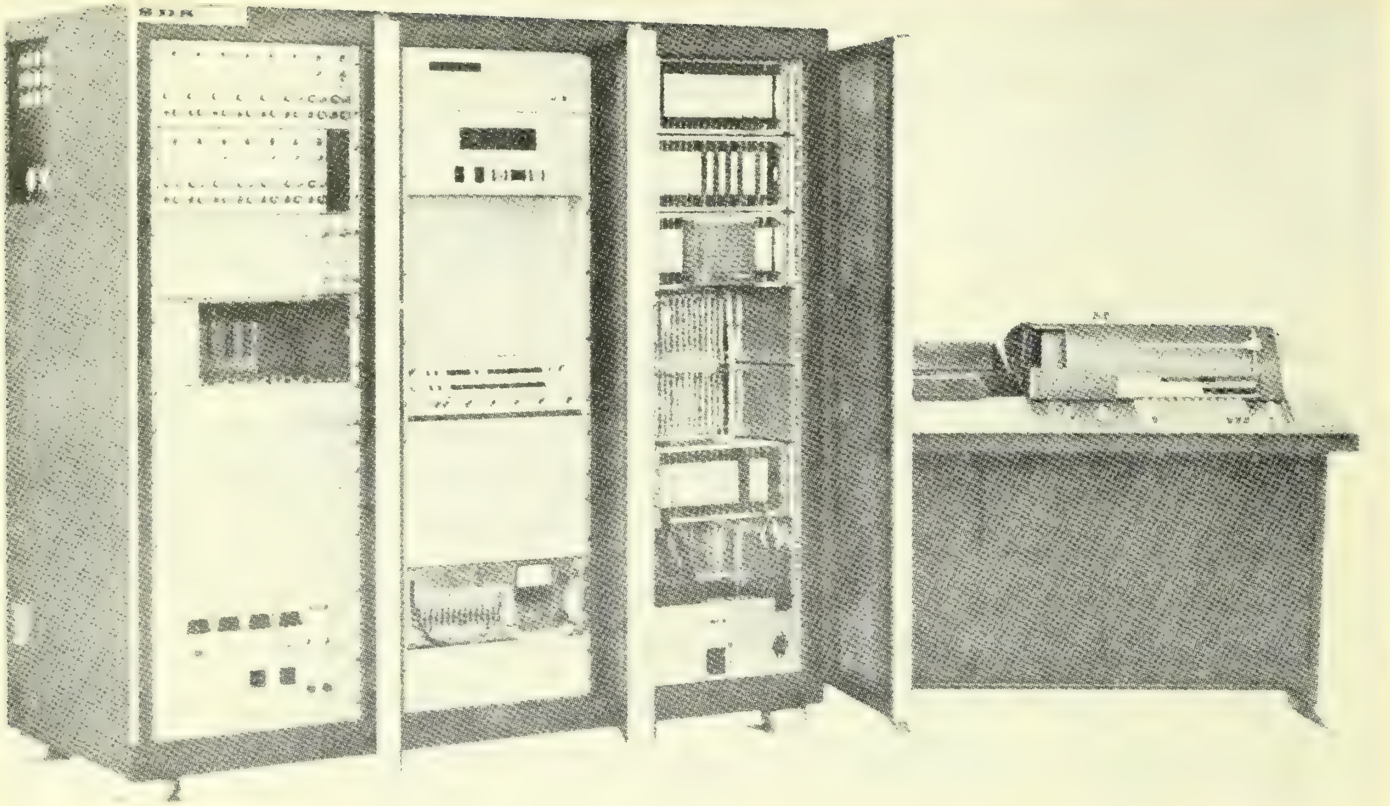


Figure 1.—Strain-history-data acquisition system.

tical nature of the problem requires that a large mass of data be collected. Enough information must be gathered on various types of bridges in representative parts of the country so that the loading histories described will be representative of what really takes place.

The Public Roads staff, having been active in the field testing of highway bridges since 1953, is well aware of the problems involved in the data reduction process and has shared in the concern of the need for better instrumentation. Accordingly, the Structures and Applied Mechanics Division of the Office of Research and Development assembled a set of specifications and employed a contractor to develop an instrumentation system to *monitor, digitize, and record physical phenomena on highway bridges in service*. Work on the project began in the summer of 1965, and the system was completed in May of 1966. Since then it has been subjected to an extensive acceptance test and used in a pilot study on a bridge near Dumfries, Va.

Description of Instrumentation

Detailed components of system

The data acquisition system is an assemblage of instrumentation consisting largely of signal conditioning modules, amplifiers, an analog-to-digital converter, a digital processing unit (computer), and an input-output device [illegible] with paper punch). Power supplies, [illegible] regulator complete [illegible] and the teletype, all units are housed in shock-mounted cabinets. The entire assembly is housed in an office trailer—not part of the development contract—that can be easily transported. Heating and cooling units automatically control the temperature and humidity in the trailer. A view of the instrumentation is shown in figure 1, and a system block diagram is presented in figure 2.

Although electric power to the system can be supplied from portable AC generators, power from the more reliable commercial power lines is preferable. A single-phase, three-wire power source of approximately 40 amperes is required. Except for the air cooling unit in the trailer, which requires 220 volts, all equipment operates on 110-volt power.

Signal conditioning modules

The transducer input circuits from the bridge structure being measured are completed in the signal conditioning modules. Inputs from one-, two-, or four-arm Wheatstone bridges are acceptable. Also, a variety of interchangeable completion cards are available for use with transducers other than strain gages, such as thermocouples, thermistors, and potentiometers. Adjustments have been provided for balancing the signal conditioning circuits and for calibration.

Amplifiers

The DC amplifier multiplexers are interchangeable units, specially designed for use in multichannel, low-level, data acquisition systems. Each unit consists of an input amplifier, modulator, demodulator, isolator, wide-band output amplifier, extra output amplifier with an active filter, and multiplexer circuit. The input range is ±10 millivolts through ±10 volts for a full-scale output of ±10 volts.

Access to three outputs is provided. From one output, the filtered, multiplexed signal is fed to the analog-digital converter. The other two outputs provide a continuous signal, which may be used for auxiliary analog recording or monitoring with oscilloscopes, tape recorders, or oscillographs.

Analog-to-digital converter

Connected to the output of the multiplexed amplifiers is the analog-to-digital converter, commonly called the digitizer, which accepts analog voltages within a range of from +10 to −10 volts and generates parallel digital output signals. The output consists of 12 binary digits, one of which is used as the sign bit. The remaining 11 are used to represent the magnitude of the analog voltage. The conversion rate is about 70,000 12-bit words per second.

Mounted just below the digitizer in the center cabinet is a visual display of the data, where the binary information from the digitizer is shown in octal form for any selected channel. The visual display is of help when the circuits are being balanced and calibrated, as separate voltmeter or ohmmeter readings are unnecessary.

Data processing unit—computer

The heart and brain of the instrumentation system is the SDS 910 computer. It accepts the raw data from the digitizer, processes and stores information, enables data to be printed out, and controls the sequencer and, thereby, the sampling rate. It has a basic core memory of 4,096 words, which could be expanded by adding additional memory modules. The word size is 24 binary digits. Four hardware priority interrupts are provided. The language used for programing is a symbolic language, but conversion routines are available so that FORTRAN II may also be used. The computer has buffered input-output capabilities at rates in excess of 60,000 characters per second, simultaneous with computation.

Operations in the system are controlled by a clock, which is a 50 kHz. tuning fork oscillator divided down by the timing generator to the appropriate frequencies used throughout the system. The timing generator emits a one-pulse-per-second interrupt signal for the computer to update the elapsed time counters.

Input-output device

Communication with the computer is achieved by a teletype machine with paper tape reader and punch placed on line. The maximum speeds of the teletype reader and punch are 10 characters per second each. Usually the program is read in with the tape reader, and specific instructions, for which provisions have been made in the program, are typed in on the teletype. Manual access to the computer is also possible through control switches on the face of the computer panel.

Computer Program

Development of the system also included the writing of a program for the gathering of strain history data on highway bridges. This program was subsequently revised and is described in detail in the succeeding paragraphs.

Program objectives

The principal objective of the program was to count the number of times that a maximum strain range occurs. A *maximum strain range* is defined as the maximum difference in strain that is caused by the passage of a single vehicle. Ten channels are monitored and a counting table for the various strain ranges is maintained for each channel.

The strain ranges are computed by establishing strain levels, as shown in figure 3. The strains on the bridge are presented to the system as analog voltages, usually in the millivolt range. They are conditioned and amplified to a maximum range of ±10 volts, full scale, and are subsequently converted to digital notation of ±2047, full scale.

The strain levels are selected by the operator and represent a percentage of full scale. The program allows for 10 strain levels per channel. When a peak, such as point A in figure 3, is detected, a search is made to determine in which level the point is located. The same procedure is then followed for a valley, such as point B. The maximum strain range is computed as the difference of the strain levels that contain the peak and valley—not the actual values of peak and valley.

To avoid the undesirable counting of peaks and valleys caused by passenger cars and other light vehicles and those caused by small oscillations of the bridge itself, a *test level*, shown as the dashed line in figure 3, is established by the operator. A peak is not recorded

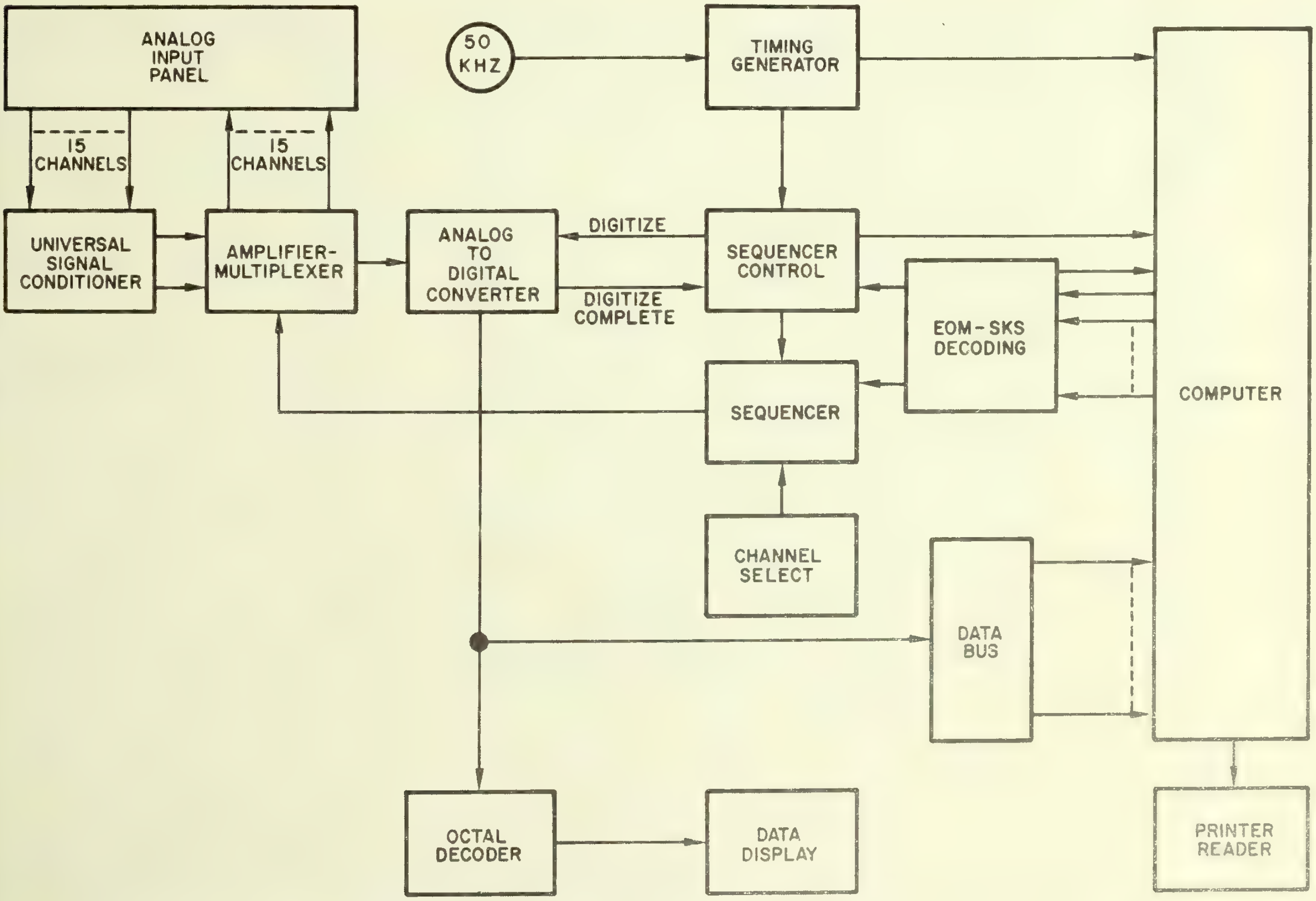


Figure 2.—Block diagram, data acquisition system.

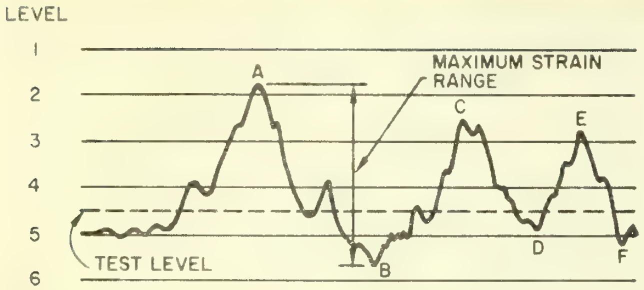


Figure 3.—Maximum strain range.

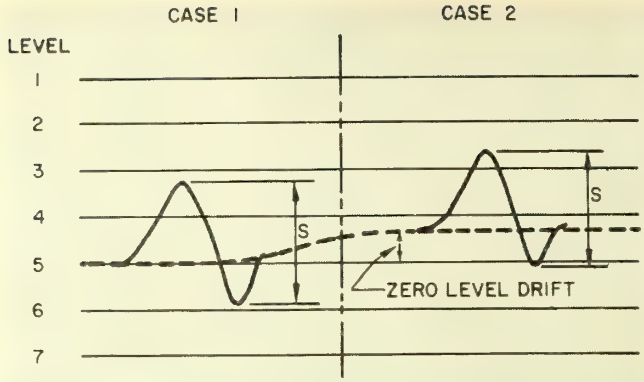


Figure 4.—Zero level drifting.

unless the occurrence exceeds this test level. To be certain that the maximum strain range is obtained, a corresponding valley is not recorded until the strain drops below the zero level.

Another objective of the program was to separate live-load strains from those caused by environmental conditions. It was intended to continuously monitor strains at any particular bridge for periods of several days. During this time the bridge will experience numerous cycles of strain caused by temperature changes. This strain is separated from live-load strains by a zero level adjustment, which is periodically performed.

When the operator determines the strain levels to be used for each channel, he also determines which of these levels will be the zero strain level. During the time that the system is sampling the 10 channels, some drifting of this zero level occurs, as shown in figure 4. To correct for this drifting, each channel is periodically scanned for its new zero level, which is found by sampling until a relatively flat curve—continuous samples of very close magnitude—is detected for a continuous, specified, length of time. This time can be varied by the operator to fit a particular traffic situation. The average value of this flat curve is computed as the new zero level. All strain levels are then shifted by the amount that the zero level shifts from its previous value.

The frequency at which the zero level adjustment is to be performed is determined by the operator when he initiates the program. Five zero level adjustments are allowed during each recording interval. As the digital value of each zero level is printed out at the end of each recording interval, the slowly varying strains in a member caused by temperature changes can be reconstructed merely by converting the values to strains.

Data acquisition and processing

The program is designed to accept and process 15 channels of low-level transducer information. Ten of these channels, primary channels, are monitored for associated peak and valley counts as described earlier. The remaining five channels, secondary channels, may be used for monitoring other transducer signals, such as thermocouples.

The samples from the 10 primary channels are processed first, beginning with channel No. 1. After processing the first sample, the program advances the sequencer to the next channel. Each sample is checked to see whether it is larger than the test level. (See fig. 4.) If it is, it is checked to see whether it is a peak. If so, the level containing the peak is recorded. If another, larger peak occurs before the signal drops below the zero level, the larger peak is recorded in place of the smaller one.

When the signal drops below the zero level, the program starts searching for a valley. It continues to search for valleys until the signal again becomes larger than the test level. At this time, the most negative strain level containing the valley is recorded, and a count is made in the appropriate storage location corresponding to the peak and valley just recorded. This process continues throughout the recording interval.

When the real-time clock detects that it is time for a zero level adjustment, the system processes the zero level adjustment program, which uses the interrupt system to obtain the samples. All 15 channels are sampled in this operation. The samples from each of the channels are accumulated, and the average value computed and saved for output. The zero level adjustment is then performed.

Program summary

The program presents a reasonably accurate method for detecting the maximum strain ranges that occur in a stressed member under cyclic live loading. For each such occurrence, a count is made in the proper matrix accumulator. The counts are accumulated for each channel separately for a predetermined length of time.

The accuracy of the maximum strain ranges recorded depends on several factors. One is the frequency of the cyclic live loading. The maximum sampling rate for each channel is set at 200 samples per second. As the response frequency increases, accuracy decreases. Good accuracy can be obtained with frequencies below 20 cycles per second, and excellent accuracy can be obtained at frequencies below 10 cycles per second.

Another factor that influences the accuracy of the maximum strain ranges is drifting of the zero strain level. The program, by performing a zero level adjustment at predetermined intervals, presents a method for separating and determining strains in members caused by environmental conditions. But between intervals, a peak and corresponding valley could be recorded in different levels, owing to the shift in the zero level by the two cases. As shown in figure 4, this could also affect the maximum strain range. In both cases, S is the same, and in both cases the valley will be recorded in level 6, but in case 1, the peak will be recorded in level 4, and in case 2, in level 3. Therefore, the maximum strain range will be recorded differently in each case.

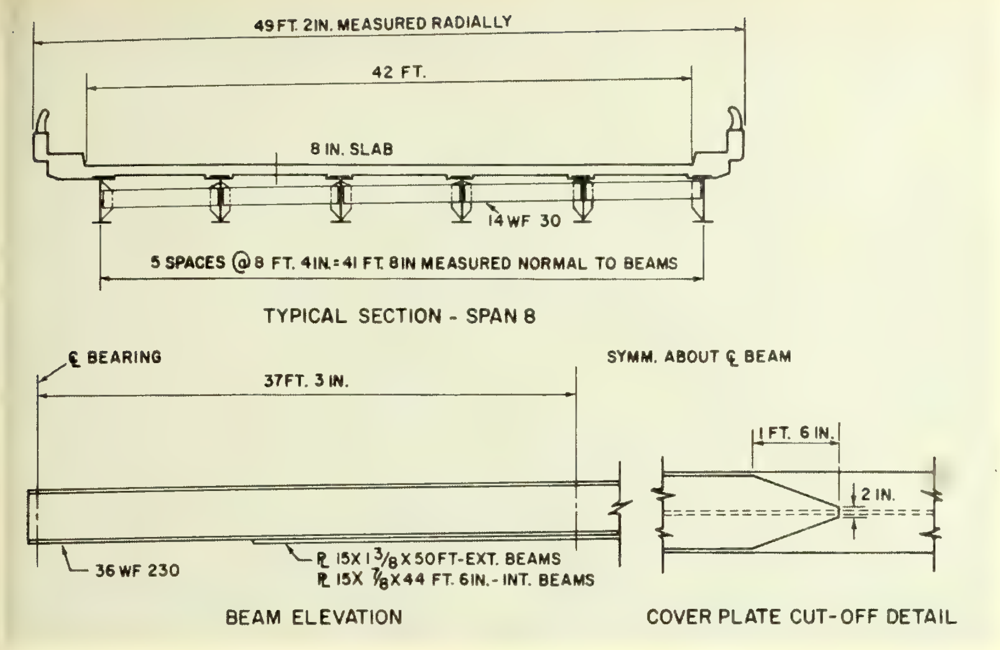
To minimize this difficulty, the zero level adjustment should be performed frequently, and the expected rate of change of the zero level must be considered when setting the length of recording time.

The accuracy of the maximum strain range is also affected by the magnitude of the difference in the strain levels that are chosen. The smaller the difference, the greater will be the accuracy.

One other source of measurement error could stem from the use of the test level, which was established to eliminate small oscillations of the bridge. The program does not search for a peak until the signal exceeds the test level. It searches for a valley only when the signal drops below the zero level. Consequently, erroneous peak and valley associations could occur infrequently. Referring to figure 3, peak C would be associated with valley F, since valley D did not drop below the zero level. Such a situation could occur when one vehicle is following closely behind another.

All these sources of error are considered minor, or can be corrected by a careful adaptation of the sampling scheme to a specific local traffic and environmental condition.

The data acquisition system and the computer program were tested on an actual bridge in a pilot study to determine just how



49 FT. 2 IN. MEASURED RADIALLY
42 FT.
8 IN. SLAB
14 WF 30
5 SPACES @ 8 FT. 4 IN. = 41 FT. 8 IN MEASURED NORMAL TO BEAMS
TYPICAL SECTION - SPAN 8
℄ BEARING
SYMM. ABOUT ℄ BEAM
37 FT. 3 IN.
1 FT. 6 IN.
2 IN.
36 WF 230
℞ 15 X 1 3/8 X 50 FT - EXT. BEAMS
℞ 15 X 7/8 X 44 FT. 6 IN. - INT. BEAMS
BEAM ELEVATION
COVER PLATE CUT-OFF DETAIL

Figure 5.—Dumfries bridge, northernmost span.

well they perform the functions for which they were designed. The conduct of the test, the results obtained, and an evaluation of the system, make up the remainder of this article.

Dumfries Field Study

The bridge chosen for the pilot study was a steel girder structure with partial-length, welded cover plates. It was a simple, four-span bridge of composite design located in the northbound lane of Interstate Highway 95 near Dumfries, Va. This site was chosen primarily because of the heavy truck traffic it accommodates, and also because of its nearness to a truck weighing station. Views of the structure are shown at the beginning of this article. The bridge has a slight skew angle, left forward at 3°46′.

Of the four spans, only the northernmost span was instrumented and tested. Structural details of this span are shown in figure 5. Ten strain gages were placed on the bottom flanges of five of the girders. The sixth girder, being the outside girder on the left side of the bridge, would have received little stress from truck traffic and was not instrumented.

On the instrumented girders, one gage was placed in the center of the bottom of the cover plate at midspan on a skewed section. Another was placed on the bottom of the girder 4 inches off the south end of the cover plate.

During the period of testing, strains were continuously monitored, the sampling intervals ranging from ½ to 2 hours. At the end of each sampling interval, the accumulated data was printed out as strain level peaks and associated valleys. From the summation of these periodic outputs, the stress range distribution for each instrumented member of the bridge was determined, as shown in figure 6. Values from beam 4 are not shown because the gages on the beam malfunctioned during part of the sampling periods.

The data in figure 6 were recorded from May to November 1967 during scattered sampling intervals. The frequency of occurrence of each stress range, computed from the total number of occurrences at each stress level for each beam, is shown in table 1. Beams were numbered right to left looking north. The maximum stress range, between 3,300 and 3,700 p.s.i. occurred on beam 3. However, considering the total number of stress range occurrences above 1,650 p.s.i., there was a larger number on beam 2 than on any other beam. Midspan stresses are shown in figure 6, but end-of-cover-plate stresses were about the same as these.

The number of occurrences shown in table 1 relates to the number of vehicles that crossed the structure, as follows. In addition to the absolute maximum stress range excursion, certain vehicles produced complementary stress cycles, which were also counted. Some of these complementary cycles could have had greater amplitudes than the absolute maximum induced stress caused by other vehicles. For example, figure 3 could have represented the stress caused by the passage of one vehicle. However, two occurrences would have been recorded—A–B and C–F. By keeping a record of the number of trucks that crossed the bridge during the time that strain ranges were recorded, it was possible to obtain the average number of occurrences per vehicle for each beam. The values for this bridge (table 1), ranged from 1.7 for beam 5 to 3.0 for beam 3. Beam 3 had not only the highest recorded stress range, but also the highest ratio of occurrences per event.

Truck type and weight distribution

A State-operated weighing station was located approximately 2 miles from the Dumfries bridge. This station, in continuous operation throughout the year, made it possible to obtain accurate truck counts. The monthly and yearly truck-traffic totals for 1966 and 1967 are shown in table 2. The

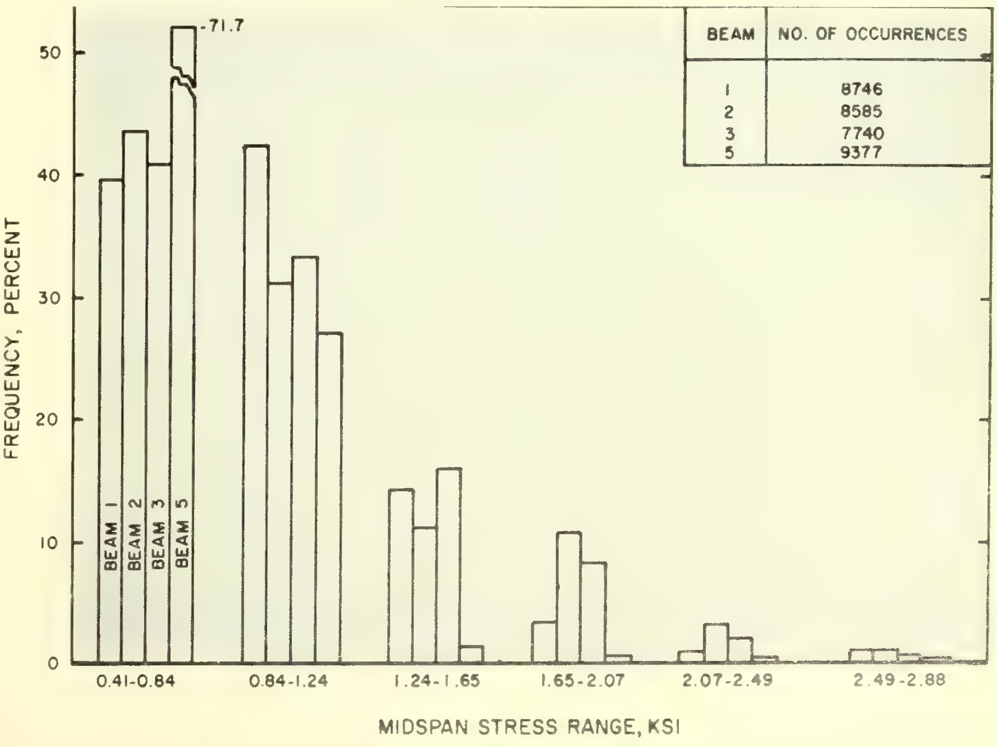


Figure 6.—Frequency distribution of stress ranges at midspan.

highest volume of truck traffic occurred during the summer months. The Virginia truck law allows a maximum gross weight of 70,000 pounds, but there is an unofficial enforcement tolerance of 5 percent, which would raise the maximum gross weight to 73,500 pounds. The truck traffic included two 5-axle vehicles, distributed as shown in figure 7.

Records of truck weights were collected at the weighing station during scattered intervals while strains were monitored. From these data, the gross truck weight distribution was determined as shown in figure 8. The greater percentage of truck traffic weighed between 20 and 30 kips, with only 5.6 percent above 70 kips. The average gross weight of each axle group is given in figure 7.

Life expectancy

An attempt was made to predict the life expectancy, or fatigue life, of the Dumfries bridge. The fatigue curve used was developed from the recommendations of the joint ASCE-AASHO Committee on Flexural Members (*1*).[2] For rolled beams with cover plates using ASTM A36 steel, the Committee recommended a stress range value of 9,000 p.s.i. for fracture at 2×10^6 cycles. In *House Document 354* (*2*), it is assumed that the stress range value at 200×10^6 cycles is equal to one-third the stress range value at 2×10^6 cycles. If the same assumption is used for the study reported here, the stress range at 200×10^6 cycles is 3,000 p.s.i. Based on the assumption that the stress range and cycles to failure have a linear log-log relation, figue 9 was constructed. This curve, along with Miner's hypothesis of cumulative damage, can be used to predict the fatigue life of a bridge, if the stress ranges to which it will be subjected are known. Other researchers (*3, 4*) also have developed fatigue curves similar to the one shown in figure 9.

The stress ranges recorded on the Dumfries bridge were not large enough to make a reliable estimate of its fatigue life. The highest stress range recorded, approximately 3,500 p.s.i. on beam 3 at midspan, had an occurrence frequency of less than 0.01 percent. Slightly lower stresses were recorded near the ends of the cover plates. The frequency of occurrence of all stresses on beam 3, which fell on the curve of figure 9, was only 0.06 percent. It is therefore impossible to predict the fatigue life of the bridge from these data.

Summary and Discussion

Evaluation of the data acquisition system indicates that the concept of combining transducer conditioning, amplifying, monitoring, and processing instrumentation into one system controlled by a digital computer is a very good one. The system, as assembled, is certainly able to monitor, digitize, and process the strains produced by ordinary traffic, without any loss of essential information and with a minimum of manpower in attendance.

Having a computer controlled data acquisition system allows for great flexibility in gathering and processing data. One guiding principle during the development of the program was simplicity. As several aspects of the fatigue problem on highway bridges are not known or understood, it was decided to concentrate only on the gathering of major stress ranges. All kinds of minor vibrations and fluctuations in stress could also have been recorded; but these have been shown to have no bearing on the life expectancy of highway structures, and the choice was made to ignore them.

Laboratory fatigue studies together with the development of an applicable cumulative damage fatigue theory will have to accompany loading history studies in the field.

Those aspects of the program relating to the zero level adjustment procedure are believed to be sound, and any errors caused by drifting that are not corrected by the zero level adjustment are very small.

The Dumfries field test was properly named a pilot study, as it was not a full fledged investigation into the loading history of a highway bridge. Initially, the bridge served mainly to check out the new instrumentation, but the volume of data gathered far surpasses that of other investigators who used oscillograph instrumentation to record the passage of trucks in similar studies.

For instance, in a recent Michigan study (*5*) involving eight bridges, only some 3,000 events were recorded for a single bridge. In a similar investigation in Maryland during the summer of 1967, only about 1,000 events were recorded. In the Dumfries study, the average for each of the several beams instrumented was about 9,000 stress events. Many more were recorded in the early parts of the study, but these were not reported here because the initial computer program did not clearly isolate stress ranges, recording only at which level a maximum peak occurred.

Table 1.—Number of stress range occurrences, midspan gages

Beam	Number of occurrences										
	Stress, range, p.s.i.									Total	Per vehicle
	3,700	3,300	2,880	2,490	2,070	1,650	1,240	840	410		
1				3	39	287	1,255	3,687	3,475	8,746	2.1
2			2	36	276	906	935	2,674	3,756	8,585	1.9
3		1	4	24	150	615	1,236	2,579	3,131	7,740	3.0
5				1	2	12	91	2,554	6,717	9,377	1.7

Table 2.—Number of trucks weighed, Dumfries weighing station, route I–95, north bound

Month	1966	1967
January	41,459	36,678
February	42,149	42,823
March	49,028	50,840
April	53,841	43,954
May	51,846	50,497
June	58,322	60,176
July	50,629	47,776
August	58,605	60,123
September	49,385	49,265
October	57,406	54,346
November	44,129	51,886
December	26,409	45,355
Total	583,208	593,719

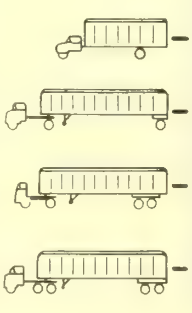
Before any conclusions can be based on the results, not only in this study but in all other similar ones, it must be determined whether the sampling of data was representative of the effects produced by the truck traffic—in 1 year for instance.

The results reported here were recorded during the following sampling periods:

May 1—½ hour, midday
2—4 hours, midday
3—3 hours, midday
July 5—1 hour, midday
6—2 hours, midday
7—4 hours, midday
October 26—4 hours, midday
27–30—64 hours, continuous run over 1 weekend
31—4 hours, midday

There was no sampling in winter or early spring, and except for the continuous weekend run, the sampling periods fell in the middle of the day, some time between 10 o'clock in the morning and 4 o'clock in the afternoon. However, it is again mentioned that other researchers in similar studies considered their sampling time, although much shorter, to be representative.

The results of the field study, as shown in the stress range histogram, figure 7, compare favorably with the results of the recent similar studies in Michigan and Maryland. An interesting comparison among nine slab and steel girder bridges can be made from the data presented in table 3. In general, the maximum measured stress ranges are well below the calculated design live-load stresses, including impact. In fact, based on the 1961



TOTAL EACH TYPE	PCT. OF OVERALL TOTAL	AVERAGE GR. WEIGHT, KIPS
223	19.9	12.5
157	14.0	28.3
332	29.7	37.7
407	36.4	45.6

Figure 7.—Truck type and weight distribution.

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[2] [illegible] the references [illegible]

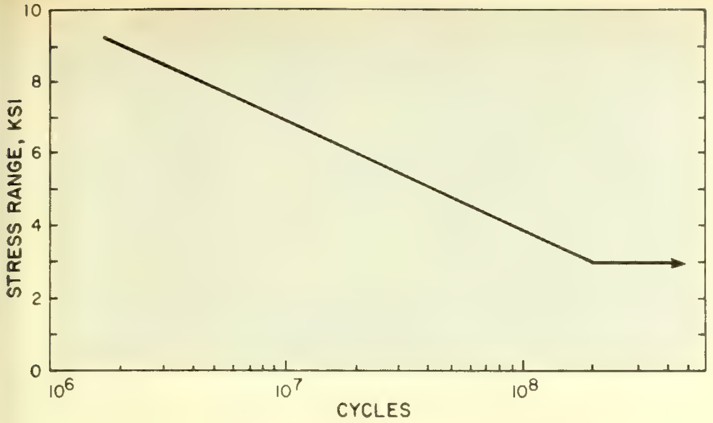


Figure 8.—Frequency distribution of truck gross weights.

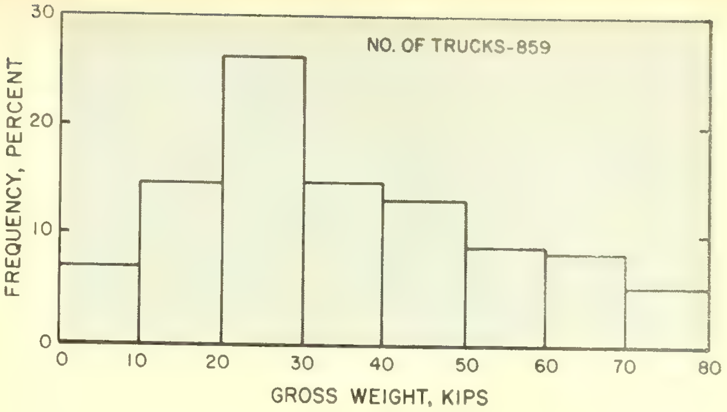


Figure 9.—Assumed S–N curve for beam with welded partial-length cover plates.

AASHO Bridge Specifications, the Dumfries maximum measured stress range was only 45 percent of the design live-load stress for an interior beam. This maximum measured stress range of 3,500 p.s.i. occurred only once in more than 8,000 events, and if the value of approximately 2,700 p.s.i. (figure 7) had been used, an even lower ratio would have resulted.

The Dumfries bridge is a 3-lane bridge with a 2½-foot sidewalk on both sides of the bridge. It is located on a rural portion of a new Interstate highway and crosses a creek as well as a small road; it is therefore at a low point in the vertical alinement of the road. During the conduct of the test it was occasionally noted that one truck attempting a passing maneuver would be exactly even with another in the centerlane, and both trucks crossed the test span at the same instant. However, most of the trucks crossed the test span singly, traveling near or above the speed limit of 65 m.p.h. Never were all 3 lanes of the bridge observed to be loaded with trucks simultaneously. As the live-load design calculations are based on 2 or more lanes being loaded simultaneously and on certain load distribution factors, the low recorded stress ranges can be explained in part by the absence of such simultaneous loading. This situation, of course, is peculiar to this structure only. Closer agreement between calculated and recorded live-load stresses would be expected on longer span, 2-lane bridges.

Because of the low recorded stresses, it was impossible to obtain a meaningful forecast of any possible fatigue damage, as pointed out earlier.

Cover-plate gages

The results from the gages that were placed 4 inches off the south end (traffic is northbound) of the cover plates were very much like the results from the midspan gages, both in distribution and magnitude. These results are not presented here.

Stresses caused by temperature

The capability of the instrumentation, with the properly programed instructions, to separately monitor and record live-load strains, as well as strains caused by temperature differentials, and perhaps moisture differentials, was described earlier. The desirability for having this capability is twofold: (1) It assures that the live-load strains are not clouded by any drifting of a baseline, and (2) it allows a monitoring of such drifting. In no way can one be certain that all the recorded drift is due to actual strains in the bridge, unless such a condition has been independently verified by another type of strain measurement.

In the Dumfries field test, extreme care was exercised to eliminate as many errors as possible. Amplifier drifting and the drifting of other instrumentation was shown to be negligible when a very stable load cell was used. Temperature compensation in the dummy gages was provided, and the dummy gages were placed as near as possible to the active gages. The one untested source of error might be due to variations in lead wire behavior during the course of a day.

It would be improper to publish any of the recorded long-term strains, as they might contain errors; but it can be said that the magnitude of the long-term strains appears to be two to three times as large as the maximum recorded live-load strain. Also, there seems to be a great variation in long-term strains between gage positions, such as at midspan and at end of cover plate on the same beam, as well as between interior and exterior beams.

Conclusions

Based on experience gained while working with the data acquisition system and on the results of the Dumfries field tests, the following conclusions seem in order:

- The data acquisition system can adequately accomplish the task it was designed to perform—to monitor and digitize strain history data on highway bridges.
- The results from the Dumfries field tests, in the form of measured stress ranges, were always less than half the calculated design live load with impact stress, and usually were much smaller than half.
- The measured stress ranges obtained from the Dumfries bridge are somewhat lower

(Continued on p. 189)

Table 3.—Stress ranges of different bridges

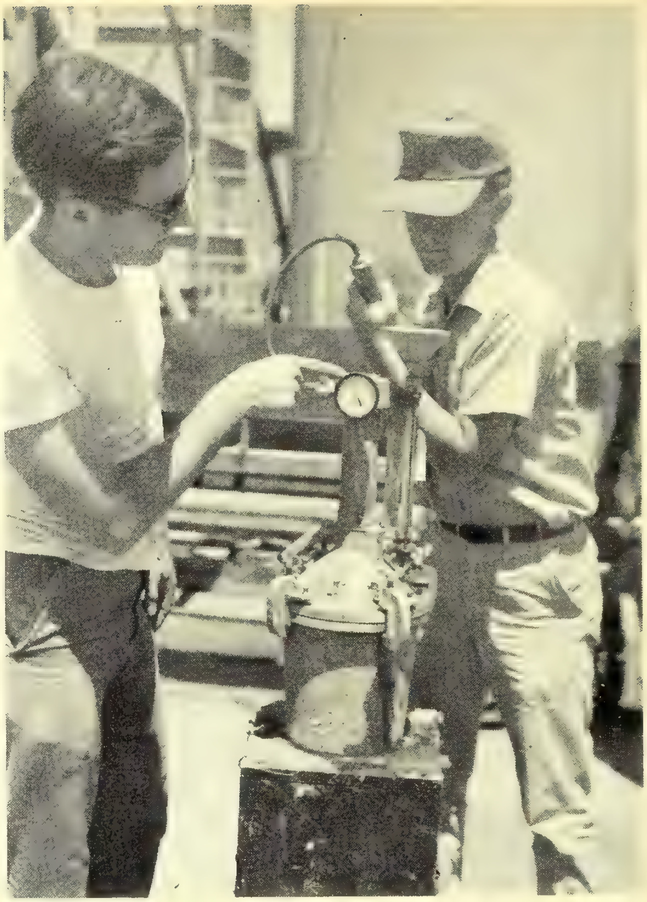
Bridge	Maximum stress range	Occurrence	Maximum measured / Design LL + I
	p.s.i.	*Percent*	
Michigan 1	6,300	0.1	$\frac{6300}{8040}=0.78$
Michigan 2	5,100	0.1	$\frac{5100}{7730}=0.66$
Michigan 3	5,100	0.3	$\frac{5100}{8500}=0.60$
Michigan 5	4,500	0.4	$\frac{4500}{7820}=0.58$
Michigan 6	3,900	0.1	$\frac{3900}{8950}=0.44$
Michigan 7	4,500	0.1	$\frac{4500}{7030}=0.64$
Michigan 8	6,300	0.2	6300 / [illegible] = [illegible]
Maryland U.S. 1	5,000	0.3	$\frac{5000}{10400}=0.48$
Dumfries	3,500	[1] 0.01	3500 / [illegible] = 0.45

[1] Occurred once out of more than 8,000

Quality Assurance in Highway Construction

Part 3—
Quality Assurance of Portland Cement Concrete

Reported by WESLEY M. BAKER, Highway Research Engineer, and THURMUL F. McMAHON, Principal Quality-Assurance Research Engineer, Materials Division



Pressure meter test for determining air content in portland cement concrete.

This is the third part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, and Part 2.—Quality Assurance of Embankments and Base Courses, were presented in previous issues of PUBLIC ROADS. The remaining parts, to be presented in succeeding issues, are 4.—Variations of Bituminous Construction, 5.—Summary of Research for Quality Assurance of Aggregate, and 6.—Control Charts.

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RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Introduction

EVER since the development of portland cement concrete for use in the construction of highway pavements and structures, highway engineers have been concerned with the quality of the concrete and of its constituents. From years of experience, methods have been developed to control the quality of concrete and measure its acceptability as a quality material. But responsibility for the quality of portland cement concrete and causes of its failure are still confusing issues.

Research has provided an insight into the causes of variations that have always existed in the results of concrete tests. It has also provided some knowledge of their magnitude as regards blending, mixing, sampling, and testing, especially under laboratory conditions. In early research it was recognized that statistical concepts afforded a useful tool to analyze the data and establish the causes of variation. In fact, as early as the 1940's, Mr. Alfred M. Fruedenthal recommended that statistical concepts be used to revise portland cement concrete specifications (*1*).[1]

A committee on quality control of concrete in the field, appointed in England at the beginning of the 1950's, reviewed all aspects of concrete production and recommended methods of improving quality and testing techniques. The committee's greatest achievement was the adoption of statistical concepts to better understand the nature of variation in concrete production. The normal distribution was shown to be applicable in the concrete industry, particularly for the cylinder strength distribution, and acceptance criteria were established at a 95-percent confidence level.

The first official action in the United States to adapt the statistical approach to quality control of concrete was taken in 1955 by the American Concrete Institute. Criteria were established and rational specifications for structural concrete were recommended.

In this report on the application of statistical quality control methods to the production of concrete by the highway industry (*2*), Mr. Edward A. Abdun-Nur advocated extensive use of statistical concepts in specification writing and acceptance sampling. He considered a realistic picture of concrete produced under normal control to be one in which the coefficient of variation of 28-day strength is 20–25 percent, and defined a good concrete as one with a 15-percent coefficient of variation.

Early work in quality control of portland cement concrete demonstrated the advantages of using statistical concepts to specify, control, and accept concrete; however, more information was needed to make optimum use of them. The Office of Research and Development, Bureau of Public Roads, has been promoting the gathering of information by the States concerning the quality of the concrete being produced under current specifications and the contributing factors in the variation of test results. The objective of this research program has not been to determine all factors relating to variations in concrete production, but to isolate variations owing to materials, sampling, and testing. The results to date are presented and analyzed in the following paragraphs.

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[1] Italic numbers in parentheses identify the references listed on p. 189.

Variability in Concrete Strengths

Strength is not always the most important characteristic of concrete quality, but it is the one that is most often measured. It is assumed to be indicative of the water-cement ratio and, accordingly, an indicator of durability. The magnitude of the variability in strength is, therefore, an indicator of the magnitude of variability of the other characteristics.

Variability in concrete strengths can be attributed to two other types of variability: (1) Inherent variability in the materials and processes that results from chance causes, which cannot be controlled, and (2) variability from assignable causes which can be controlled. The attainable quality of concrete in the field is limited not only by the chance causes that contribute to the variation in quality, but also by the economic factors entailed in reducing the assignable causes.

The measurement of variability from chance or inherent causes is complicated by the inherent variability of each of the ingredients in the mix, which can interact with the processes of blending, mixing, and placing, and result in a much larger variability in the concrete itself.

The assignable causes of variability are more numerous and more difficult to isolate, but the production of quality concrete is de-. pendent on the reduction of all variables. However, the isolation and restriction of variables can be carried only as far as economic conditions warrant. Under the present state of knowledge, the ultimate uniformity of concrete production cannot be precisely stated. Extensive research will be necessary to isolate variables and to determine the extent to which variation can be reduced. Current information can be used only to show that variation does exist, and that sampling and testing often contribute as much of the variability as do the ingredients and processes used in construction.

Mr. H. H. Newlon, in a paper (*3*) described and discussed numerous variables affecting concrete quality. The data concerning concrete variability, presented in table 1, was based on a similar tabulation from his paper. Although such information is interesting and may be used to design specification limits, it is of little worth to the overall problem of reducing variability in concrete construction. The basic need is to isolate the common factors affecting variability. Research aimed at this purpose has been underway for the past 4 years.

Data based on a West Virginia research report (*4*) are presented in table 2. These data are illustrated in figure 1, which depicts the relations among the materials, sampling, and testing standard deviations. As standard deviations are not additive, the sum of the standard deviation shown does not equal the standard deviation of the concrete strength. A significant indication from these data is that the combined testing and sampling variations are usually greater than the material variation. It is also significant that the materials deviations consist of the material and process variations, whereas the sampling and testing deviations are caused by the measurement process. Figure 1 indicates that sampling did not contribute significantly to the variation of test results on these projects.

An analysis of historical data on compressive strengths of concrete cylinders, presented in table 3, was based on a report by the State Road Department of Florida (*5*). Two types of concrete, class *A* and class *NS* were analyzed in the report. Routine control is normally exercised over class *A* concrete, whereas class *NS* concrete is spot checked only occasionally.

Based on the same Florida report the mean strength, standard deviation, and coefficient of variation for the concrete, shown in table 3, were compared by source in table 4. Researchers have shown that the standard deviations of strength test results usually increase with the mean or average strength of the concrete; therefore, the best comparison of these data may be shown by the coefficient of variation. As expected, the coefficients of variation for class *NS* concrete were greater than those for class *A*.

The strength data presented—typical of nearly all strength data received—expressed large standard deviations with average strengths well above the usual minimum of 3,000 p.s.i. For example, if the class *A* concrete of project 4, table 3, were analyzed using normal distribution methods, there would be less than 1 percent chance that a test would result in a compressive strength of less than 3,988 p.s.i. With the same standard deviation, the mean could be as low as 4,698 p.s.i. before a 1-percent chance of being below 3,000 p.s.i. was exceeded. If the standard deviation could be reduced, the mean might be lowered further without risking noncompliance. However, as pointed out previously, the durability

Table 1.—Average deviation of concrete strengths

Agency	Concrete type	Data source	Average standard deviation [1] $(\bar{\sigma})$
			p.s.i.
Bureau of Public Roads	Paving	Research	585
Do	do	Historical	473
Do	Structural	do	576
Virginia Department of Highways	do	Special	467
Do	Paving	Cores	663
Ontario Department of Highways	do	Routine	494

[1] The average standard deviation for all data presented.

Table 2.—Portland cement concrete variations

Project No.	28-day compressive strength variations								
	Mean $(\bar{X})$	Overall standard deviation (σ_o)	Overall coefficient of variation (v_o)	Standard deviation, testing (σ_t)	Coefficient of variation, testing (v_t)	Standard deviation, sampling (σ_s)	Coefficient of variation, sampling (v_s)	Standard deviation, materials (σ_m)	Coefficient of variation, materials (v_m)
Structural concrete									
	p.s.i.	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*
1	4,235	435	10.0	170	4.0	236	5.6	310	7.2
2	4,420	482	10.9	323	7.3	39	0	360	8.1
Paving concrete									
1	4,675	545	11.7	377	8.1	91	0	386	8.3
2	3,755	420	11.2	322	8.5	42	0	270	7.1
3	3,720	575	15.5	318	8.5	-----	0	495	13.3
4	4,760	467	9.8	200	4.2	34	0	420	8.8
5	4,688	733	16.5	585	12.5	-----	0	545	11.7

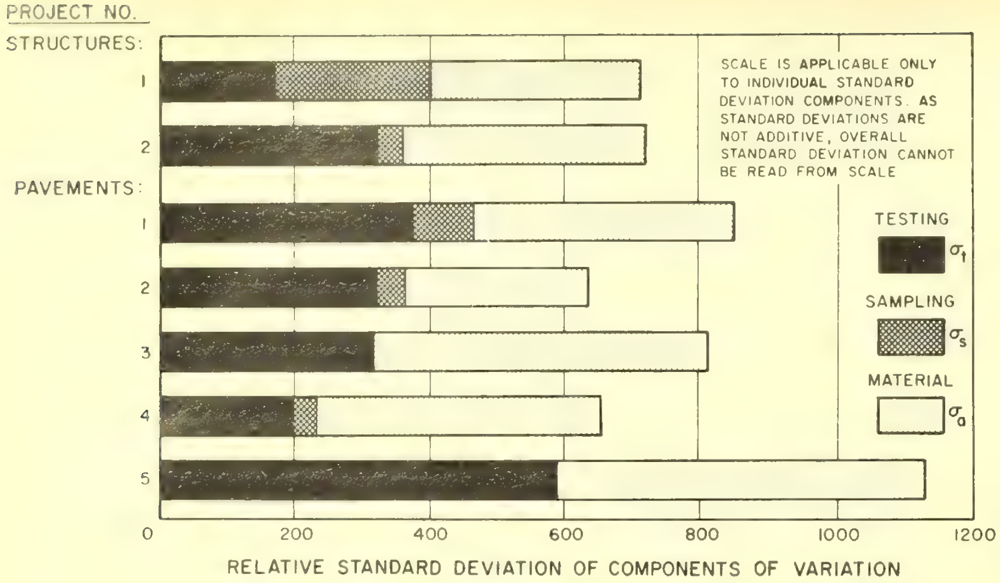

Figure 1.—Portland cement concrete—standard deviation, compressive strength.

of the concrete may be the controlling factor in reducing strength through lowering the design cement content. If it is assumed that strength is an indicator of water-cement ratio, it is possible that more uniform strength will also result in a more uniformly durable concrete. This in turn may allow a reduction of the design cement content.

If durability should be the controlling factor and if it should have a relation with strength, a test for durability should be developed. This test would eventually replace the strength test in measuring concrete quality.

Variability of Plastic-Concrete Air Content

Air content of portland cement concrete is one of the most important factors in the durability of pavements and bridge decks. Not only is it important that the air content be sufficient to prevent damage from freeze and thaw cycles and low enough to preserve strength, but it is also important that the air be distributed uniformly throughout the mix.

Several States have studied variations in air content and evaluated the performance of different test methods. The data shown in table 5, submitted by the State of New York, are representative of this research. The tests proved that, in the State of New York, truck mix concrete was more variable with respect to air than was [illegible] mix or central mix concrete. The tests also showed that the air content measured by the Chace meter was considerably higher than that measured by the pressure meter. As tests with the Chace meter are faster than those with the pressure meter, considerable interest exists in determining the number of Chace meter tests that would give an average that has the same degree of precision as an average based on a lesser number of pressure meter tests.

The equivalency of the Chace and pressure meters can be determined by a comparison of the sampling and testing variance of each in the following manner:

$$\frac{\sigma_{st}^2(C)}{n}=\frac{\sigma_{st}^2(P)}{1}$$

In project 2, table 5, this results in the following equivalency:

$$\frac{.354}{n}=\frac{.166}{1}\ \therefore n=2.1$$

or

2 Chace tests = 1 pressure test

Equivalencies computed for the other projects shown in table 5 ranged from one to 20; six of the 10 were below four, indicating that averages based on four Chace meter tests may be suitable for control purposes. However, the wide variation in these results indicate that the actual equivalency of the two tests depends somewhat on the operator's

Table 3.—Historical concrete strength data

	Samples	Mean ($\overline{X}$)	Overall standard deviation (σ_o)	Coefficient of variation (v)	Testing error (σ_t)
28-day cylinders—class *A* concrete					
	Number	*p.s.i.*	*p.s.i.*	*Pct.*	*p.s.i.*
Project number:					
1[1]	536	4,524	396	8.8	175
2	292	4,881	540	11.1	207
3	96	5,686	544	9.6	185
4	192	5,527	566	10.2	155
5	196	5,098	577	11.3	[2]48
6	112	4,826	608	12.6	196
7	258	5,469	667	12.2	150
8	320	5,244	674	12.9	158
9	232	5,289	711	13.4	192
10	176	4,927	725	14.7	210
11	126	5,067	732	14.4	162
Average[3]	230	5,140	613	13.1	179
Range	224	860	192	5.1	60
28-day cylinders—*NS* concrete					
Project number:					
1[1]	50	4,021	[1]398	9.9	115
2	340	3,555	550	15.5	93
3	240	4,006	580	14.5	134
4	200	3,474	605	17.4	122
5	240	3,781	670	17.7	96
6	196	4,192	729	17.4	160
7	148	4,213	733	17.4	92
8	94	4,239	774	18.3	87
9	108	3,657	774	21.2	116
10	182	3,674	776	21.1	113
11	156	4,110	776	18.9	70
12	224	4,179	825	19.8	127
13	138	3,941	884	22.4	161
Average[3]	178	3,926	698	17.8	114
Range	246	765	334	7.9	91

[1] Values not included in range calculations.

[2] Statistical outlier—not included in calculation of range or average.

[3] Averages are not weighted and include all values except the outlier.

Table 4.—Production source comparison, historical concrete strength data

Source	Concrete class	Samples	Mean strength ($\overline{X}$)	Pooled standard deviation ($\overline{\sigma}$)	Coefficient of variation (v)
		Number	*p.s.i.*	*p.s.i.*	*Pct.*
1	*A*	480	4,799	593	12.4
	NS	218	4,091	620	15.2
2	*A*	540	4,906	672	13.7
	NS	360	4,210	660	15.7
3	*A*	440	5,054	585	11.6
	NS	308	4,031	584	14.5

Table 5.—Air content of plastic concrete, research data

Project No.	Mixer type and use	Observations	Test method	Testing variance (σ_t^2)	Sampling variance (σ_s)	Material variance (σ_a^2)	Standard deviation (σ)	Mean ($\bar{X}$)
		Number		*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	Central mix	216	Pressure	0.043	0.07	0.80	0.95	4.55
	Paving	216	Chace	0.20	0.545	0.65	1.18	6.39
2	Central mix	200	Pressure	0.126	0.04	0.50	0.82	5.91
	Paving	200	Chace	0.22	0.134	0.70	1.026	7.42
3	E-34 paver	200	Pressure	0.067	0.05	0.48	0.77	5.14
	Paving	200	Chace	0.15	0.10	0.33	0.76	7.38
4	Truck mix	200	Pressure	0.16	0.29	1.74	1.48	7.90
	Structural	200	Chace	0.335	0.15	1.38	1.36	10.2
5	Truck mix	204	Pressure	0.035	0.04	1.27	1.16	6.11
	Structural	204	Chace	0.45	0.99	0.48	1.39	8.77
6	Central mix	200	Pressure	0.06	0.105	0.34	0.71	6.18
	Paving	200	Chace	0.256	0.137	0.40	0.89	7.39
7	E-34 paver	200	Pressure	0.08	0.08	0.39	0.74	4.34
	Paving	200	Chace	0.14	2.55	−0.96	1.32	6.41
8	E-34 paver	200	Pressure	0.047	0.04	0.70	0.89	4.82
	Paving	200	Chace	0.26	0.25	1.02	1.24	6.51
9	Truck mix	200	Pressure	0.05	0.135	2.39	1.60	5.80
	Paving	200	Chace	0.43	0.325	1.79	1.60	8.43
10	Truck mix	200	Pressure	0.09	0.136	1.64	1.37	6.07
	Structural	200	Chace	0.28	0.24	1.71	1.49	6.33

dexterity; and, consequently, it may be necessary to establish operator equivalencies to provide sufficient confidence for the control of air content by the Chace meter in any test.

The data on air content presented in table 6 was reported by West Virginia (*4*). The data indicate that in measuring air content, there is good agreement between the Chace and Roll-A-Meter. Calculation of the equivalency of the two tests indicates that in West Virginia, four Chace tests will adequately substitute for one Roll-A-Meter test.

Variability of Concrete Consistency

The consistency of plastic concrete, as measured by slump cone or Kelly Ball tests, is a measurement of the workability of the mix and an indicator of the water content. However, consistency is no direct measurement of the water content, as air, gradation, and temperature also affect the consistency. The results of these tests therefore are a good indicator of the uniformity of the mix, and relate to a combination of these factors rather than to any one of them.

Results of studies by several States of concrete consistency, as measured by the slump cone, are presented in table 7. The data indicate little difference in the variability in slump among the several methods of concrete production, although there is considerable difference from project to project. The data also indicate that actual material variation contributes more to the overall variation than do sampling and testing.

Data from reports by West Virginia (*4*) and California (*6*) on studies to evaluate the Kelly Ball test are shown in table 8. These data and that from other sources indicate that the Kelly Ball test is valid for measuring the consistency of concrete when three readings are averaged to obtain one test value as required in the standard method.

Table 6.—Portland cement concrete pavement air content, research data

Project No.	Observations	Test method	Testing variance (σ_t)	Sampling variance (σ_s^2)	Material variance (σ_a^2)	Standard deviation (σ)	Mean (X)
	Number		*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	176	Roll-A-Meter	0.102	0.133	1.352	1.21	5.8
	141	Chace	0.334	0.233	0.565	1.02	5.06
2	104	Roll-A-Meter	0.109	0.608	0.000	0.83	5.8
	104	Chace	0.470	0.710	0.000	1.00	5.44
3	200	Roll-A-Meter	0.153	0.362	1.042	1.24	5.1
	196	Chace	0.531	0.769	0.913	1.30	5.16
4	172	Roll-A-Meter	0.126	0.248	0.191	0.71	5.1
	154	Chace	0.271	1.026	0.08	1.09	4.85
5	192	Roll-A-Meter	0.143	0.110	1.229	1.16	5.0
	198	Chace	0.249	0.148	1.36	1.32	5.54

Table 7.—Variability of concrete consistency, slump cone method

Project No.	Observations	Testing variance (σ_t^2)	Sampling variance (σ_s^2)	Material variance (σ_a^2)	Overall standard deviation (σ)	Mean ($\bar{X}$)	Specification limits
State 1							
	Number	*Inch*	*Inch*	*Inch*	*Inch*	*Inches*	*Inches*
[1] 1	184	0.16	0.04	0.26	0.68	2.44	0.5–3.5
[2] 2	200	0.13	0.02	0.45	0.80	1.5	0.5–3.5
[2] 3	300	0.25	0.09	0.46	0.89	2.76	0.5–3.5
State 2							
[3] 1	216	0.074	0.00	0.15	0.47	2.04	
[4] 2	200	0.06	0.06	0.37	0.70	1.86	
[5] 3	200	0.08	0.025	0.42	0.73	2.34	
[5] 4	200	0.027	0.012	0.206	0.495	1.77	
[6] 5	204	0.066	0.03	0.305	0.633	2.37	
[6] 6	200	0.033	0.034	0.14	0.456	2.12	
[7] 7	200	0.084	0.086	0.20	0.609	2.41	
8	200	0.158	0.047	0.50	0.844	2.26	

[1] Pavement concrete, truck mix.
[2] Pavement concrete, truck mix, slipform.
[3] Pavement concrete, central mix, screw spreader.
[4] Concrete base, central mix, slide spreader.
[5] Structural concrete, truck mix.
[6] Pavement concrete, central mix, slipform.
[7] Pavement concrete E-34 paver.

Aggregate Size Variation

One factor stressed in concrete specifications and in concrete-production control is gradation of coarse and fine aggregates. Research shows that, within projects and between projects, there is considerable variation in size distribution of aggregates in the mix. In fact, aggregates-size-distribution specifications are seldom complied with. Part 5 will contain a detailed report on the gradation of concrete aggregates, but table 9 is included here to illustrate the variation. The data from project 1 indicate little variation of either material or of sampling and testing; however, the data from projects 2 and 3 indicate a large material variance, and it is probable that the specification limits were exceeded on many individual tests.

Variability in Pavement Thickness

Pavement life expectancy is based on estimated traffic and pavement design thickness. There is still argument as to whether the design of the pavement should be based on minimum thickness or average thickness; however, as in all structures, stresses are concentrated at the weaker points, and it is axiomatic that large variations in thickness are detrimental to the pavement. Uniformity of thickness will promote better slab action and, therefore, prolong pavement life.

Variation in concrete pavement thickness is shown by the data of figure 2, which is based on a report by the State of Michigan (*7*). The data depicted represent 656 cores taken from 15 projects from 1959–61. The historical data in table 10, extracted from a report by Louisiana (*8*), substantiates the variations shown in figure 2.

The variation shown in table 11 are from a statistical study of pavement thickness by the State of Oklahoma (*9*). The thicknesses were measured by a probe inserted in the plastic concrete. Little variation was exhibited in project 3; but as the mean was below the specified thickness, the pavement life expectancy was less than desired. Project 3 had a probable range of thickness from 7.8 to 10.2 inches, resulting in weak areas that would probably reduce the life of the pavement.

According to the high average concrete thicknesses reported in the Michigan and Louisiana studies, an excess of concrete is being placed by the contractors to avoid penalties. This same high variability–high average thickness relation has also been reported in other studies of thickness. Better control of placement not only could provide savings in concrete, but also produce pavement that is capable of better performance. Moreover, the development of a standard method for measuring the depth of plastic concrete, as placed, would aid in the control of thickness and eliminate expensive coring of the hardened pavement.

Variation in Portland Cement

The production of portland cements is being closely controlled by producers, according to the historical data on chemical analyses reported by several States. It is evident that State highway departments can reduce the testing of portland cement to at least the level recommended by ASTM in section 6 of ASTM–C–183–65T.

Conclusions

Variations in what is generally considered good construction has been shown by the research summarized here. However, the variations are of considerable magnitude and could be important factors in the performance of concrete structures. An awareness of these variations is insufficient; research must be undertaken to evaluate their effect and to develop procedures by which they can be reduced.

In many test results much of the measured variation could be attributed to sampling and testing methods and procedures, and therefore the real variation may not be as large as results indicate. One of the major needs in concrete production is the development of better methods to measure the quality attributes of the concrete and the materials incorporated therein. Furthermore a clear delineation of responsibility should result in a more uniform product. It is the contractors' responsibility to produce quality material and the States' responsibility to measure the quality produced. Better measurement performance by the State highway departments will allow a more accurate estimate of product quality and provide a better basis for enforcement of the specifica-

Table 8.—Methods of measuring consistency of plastic concrete, research data

Project No.	Samples	Test	Mean ($\bar{X}$)	Standard deviation (σ)	Testing variance (σ_t^2)	Sampling variance (σ_s^2)	Material variance (σ_a^2)
			West Virginia				
	Number		*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
1	200	Slump	2.4	0.8	0.095	0.095	0.52
		Kelly Ball [1]	2.4	0.7	0.108	0.062	0.30
			California				
1	[2] 200	Kelly Ball [3]	3.69	0.91	0.08	0.14	0.61
2	[2] 200	do	3.85	0.94	0.22	0.04	0.63
3	[2] 200	do	4.00	1.27	0.13	0.10	1.39
4	[4] 200	do	1.74	0.65	0.32	0.00	0.10

[1] Convereted to inches of slump. Conversion factor—Slump inches =0.59 Kelly Ball +1.02.

[2] Structural.

[3] Converted to inches of slump by calibration—1-inch penetration of ball indicates 2 inches of slump.

[4] Pavement.

Table 9.—Analysis of variance, intermediate aggregate, percent passing ¾-inch sieve

Project No.	Range (R)	Mean ($\bar{X}$)	Material variance (σ_a^2)	Sampling variance (σ_s^2)	Testing variance (σ_t^2)	Standard deviation (σ)	Samples
	Pct	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Number*
1	79–98	92.60	4.25	0.00	8.12	3.52	200
2	33–89	69.09	122.92	5.59	4.54	11.54	200
3	34–92	71.52	124.04	9.31	24.46	12.56	200

Table 10.—Summary of statistical results on thickness of concrete pavement

	Samples	Mean ($\bar{X}$)	Overall variance (σ^2)	Standard deviation (σ)	Minimum	Maximum
			8-inch uniform thickness			
Project number:	*Number*	*Inches*	*Inch*	*Inch*	*Inches*	*Inches*
1	34	8.66	0.192	0.435	7.63	9.53
2	39	8.42	0.171	0.415	7.61	9.13
3	48	8.35	0.040	0.200	7.86	8.80
4	58	8.36	0.077	0.276	7.76	9.49
5	61	8.05	0.035	0.185	7.66	8.59
6	66	8.11	0.089	0.300	7.46	8.78
7	73	8.06	0.112	0.333	7.58	9.58
Pooled values		8.29	0.088	0.300		
			9-inch uniform thickness			
Project number:						
1	35	9.25	0.046	0.210	8.93	9.67
2	51	9.19	0.121	0.350	8.55	10.10
3	58	9.28	0.048	0.220	8.84	9.99
4	65	9.18	0.060	0.240	8.78	9.92
5	74	9.20	0.185	0.430	8.69	11.69
6	88	9.11	0.029	0.170	8.85	9.66
Pooled values		9.20	0.083	0.290		
			10-inch uniform thickness			
Project number:						
1	64	10.38	0.061	0.240	9.41	10.91
2	124	10.34	0.079	0.280	9.82	11.48
3	132	10.35	0.079	0.230	9.75	10.94
4	141	10.28	0.083	0.290	9.63	11.27
Pooled values		10.34	0.069	0.270		

Table 11.—Variation in pavement thickness, probe method

Project No.	Observations	Standard deviation (σ)	Mean ($\bar{X}$)	Specification
	Number	*Inch*	*Inches*	*Inches*
1	72	0.3	8.5	8.0
2	95	0.1	8.9	9.0
3	100	0.4	9.0	9.0

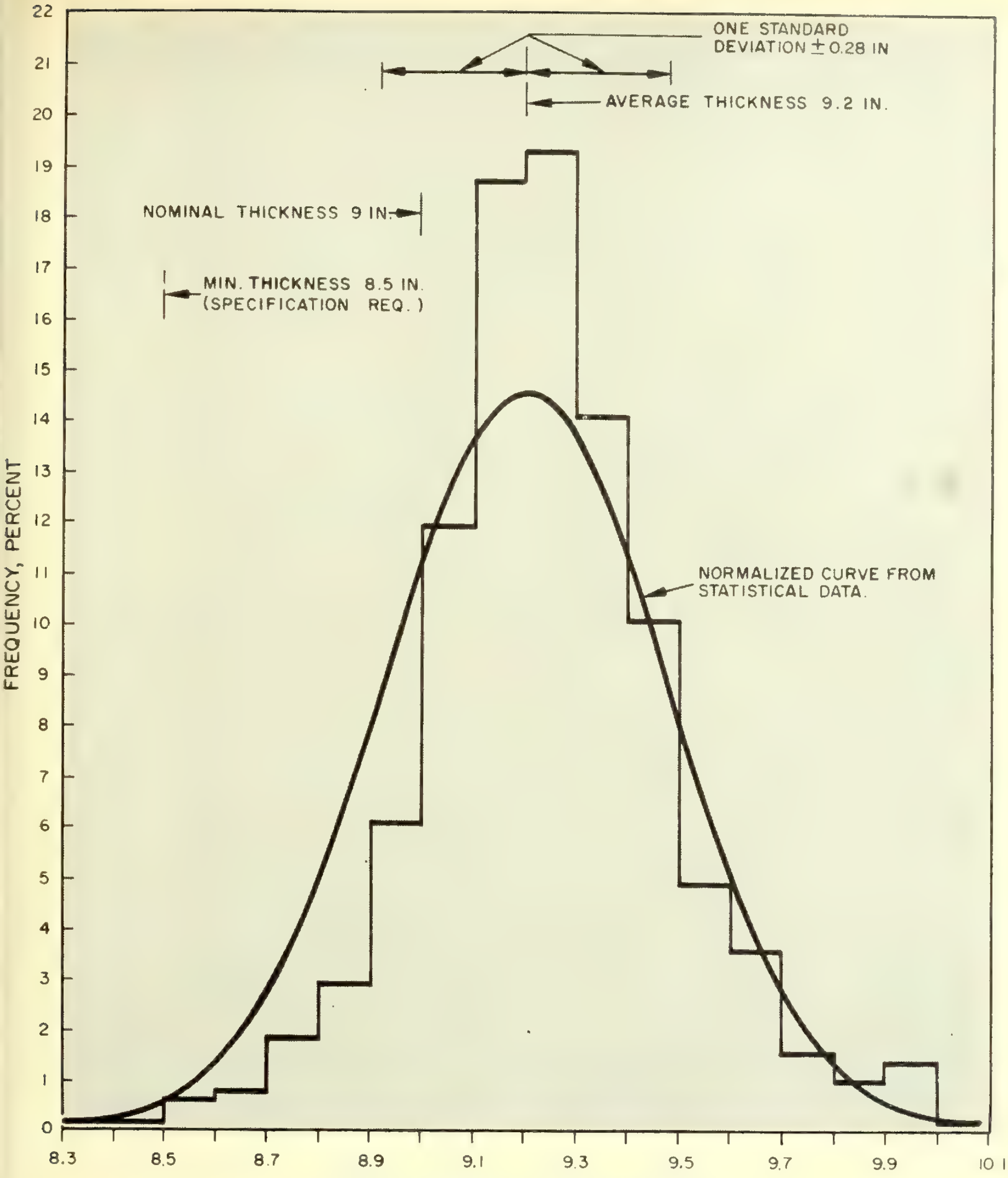

Figure 2.—Frequency distribution of concrete pavement thickness, 1959–61.

tion requirements. This approach can result only in a product that is more uniform in character and has improved performance expectancy.

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Acquisition of Loading History Data on Highway Bridges

(*Continued from p. 183*)

than measured results from other recent similar investigations, specifically those conducted on eight bridges in Michigan and on one bridge in Maryland.

- Because of the low-measured stress ranges at the Dumfries bridge, elements in that particular bridge do not appear to be in danger of fatigue failure caused by the regular truck traffic.

The last conclusion is valid only if the sampling can be termed representative of the truck traffic crossing the bridge; if it is assumed that the occasionally allowed overloads do not induce stresses in excess of the maximum recorded ones, more than 3,500 p.s.i.; and if the slowly varying stresses caused by environmental changes do not contribute to fatigue distress.

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Travel by Motor Vehicles in 1967

*BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS*

**Reported by W. JOHNSON PAGE,
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Current Planning Division**

MOTOR vehicle travel in the United States in 1967 totaled 965.1 billion vehicle-miles, an increase of 3.7 percent over the travel in 1966. The travel data were compiled by the Bureau of Public Roads from information supplied by the State highway departments. Total travel for 1968, based on information for the first 9 months of the year, is estimated at 1 trillion, 10 billion vehicle-miles, a 4.7-percent increase over 1967.

The term vehicle-miles and the other technical terms used in this article are defined in the following statements:

Vehicle-miles.—Vehicle-miles refers to the amount of travel by one motor vehicle traveling 1 mile and includes travel on all highways and streets in the United States.

Trailer combinations.—A trailer combination is a truck or truck tractor pulling one or more trailers and/or a semitrailer.

Motor-fuel consumption.—Motor-fuel consumption is the total consumption of motor fuel by highway vehicles for the year, obtained from State records.

Motor-fuel consumption rate.—Motor-fuel consumption rate is the average rate of motor fuel usage in miles per gallon (m.p.g.).

The travel and related information for 1967 are shown in table 1 by road system and vehicle type. Total travel and travel by highway system are considered to be final figures, but because of incomplete data on which to make the distribution by vehicle type, the travel by vehicle type is subject to revision. Such data have been reported in PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH for a number of years; the latest for 1965 and 1966 appeared in vol. 34, No. 12, February 1968, pages 267–269.

Travel estimates by State and administrative highway system are shown in table 2. These are based on estimates prepared by the State highway departments and reported annually to the Bureau of Public Roads, beginning in 1966.

Table 1 has been adjusted to a new base for 1967 to bring it into agreement with table 2. Main rural roads travel in table 1 includes all travel on systems shown as 01, 31, 03, 05, and 09 in table 2. Travel on local rural roads includes travel in systems 07 and 11. Travel on urban streets consists of travel on all even numbered systems in table 2. Prior to 1967, parts of systems 05 and 09 had been classed as local rural roads.

Because of their intended principal use, the State estimates of 1967 travel were made according to a system classification and rural-urban distinction directly related to the Federal-aid program. In the Federal-aid law, an urban area is "an area including and adjacent to a municipality or other urban place having a population of 5,000 or more . . ." In the annual estimates reported in table 1 in the past, however, *urban* signified the areas within the political boundaries of municipalities such as cities, boroughs, and villages.

As State estimates are now being received annually, and because it is intended to use them as a base for table 1, it was decided to redefine main rural roads, local rural roads, and urban streets so that it would not be necessary to split mileage and travel in any of the administrative highway systems.

These adjustments resulted in changes in the proportionate distribution of mileage and travel among the three highway categories in table 1. The main rural roads category changed from 35.2 percent of the total travel on 14 percent of the mileage in 1966 to 37.3 percent of the travel on 15 percent of the mileage in 1967. Local rural road travel was 14.3 percent of the total in 1966 on 72 percent of the total mileage, and 12.6 percent of the travel on 71 percent of the mileage in 1967. In 1966, 50.5 percent of the travel was on urban streets, which comprised 14 percent of the total mileage. These proportions for 1967 are 50.1 percent and 14 percent, respectively. Thus, it can be seen that the mileage shifted from urban streets to main rural roads had a much greater effect on the distribution of travel than on the distribution of mileage.

Passenger cars represented 81 percent of all vehicles registered, and accounted for 80 percent of all travel in 1967; motorcycles, 2 percent of all vehicles registered and less than 1 percent of all travel; and trucks and truck combinations, 16 percent of all vehicles regis-

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1967 [1]

	Motor-vehicle travel [2]					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural roads	Local rural roads	All rural roads	Urban streets	Total			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles gallon*
[illegible]										
[illegible]					770,971	80,458	9,582	55,220	686	13.96
[illegible]					7,737	1,953	3,962	103	53	75.00
[illegible]	273,332	94,597	367,929	410,779	778,708	82,411	9,449	55,323	671	14.08
[illegible]										
[illegible]	1,007	182	1,189	1,934	3,123	90.5	34,508	667	7,370	4.68
School	791	727	1,518	338	1,856	246.7	7,523	262	1,062	7.08
All buses	1,798	909	2,707	2,272	4,979	337.2	14,766	929	2,755	5.36
[illegible]	275,130	95,506	370,636	413,051	783,687	82,748	9,471	56.252	680	13.93
[illegible]										
[illegible]	63,221	24,426	87,647	60,021	147,668	15,364	9,611	14,491	943	10.19
[illegible]	21,617	1,400	23,017	10,760	33,777	830	40,695	6.950	8,373	4.86
[illegible]	84,838	25,826	110,664	70,781	181,445	16,194	11,204	21,441	1,324	8.46
[illegible]	359,968	121,332	481,300	483,832	965,132	98,942	9,755	77,693	785	12.42

[1] [illegible]

[2] [illegible] to agreement with table 2. Consequently, when the data is compared with that of 1966, a decrease in travel on local roads is shown [illegible] travel as percentages of total travel.

[3] [illegible] not available by highway category.

Table 2.—Vehicle-miles, by State and highway system—1967

(Millions)

Division	State	Federal-aid highway system																		Not on Federal-aid systems						
		Interstate rural			Interstate urban			Subtotal Interstate	Other primary			Secondary					Total Federal-aid rural	Total Federal-aid urban	Total Federal-aid	Other State rural	Other State municipal	Local rural	Local municipal	Subtotal rural	Subtotal urban and municipal	Total
		Final	Traveled 1/	Total rural	Final	Traveled 1/	Total urban		Rural	Urban	Total	State rural	State urban	Local rural	Local urban	Total										
		01	31		02	32			03	04		05	06	07	08					09	10	11	12			
New England	Connecticut	541	162	703	1,921	298	2,219	2,922	1,160	1,618	2,778	964	754	7	44	1,669	2,734	4,635	7,369	218	1,623	240	4,564	3,192	10,822	14,014
	Maine	495	49	544	63	67	130	674	1,367	430	1,797	817	124	-	-	941	2,728	684	3,412	856	291	278	387	3,862	1,362	5,224
	Massachusetts	1,047	174	1,221	1,289	540	1,829	3,049	2,418	4,501	6,919	611	523	1,059	1,306	3,499	5,309	8,158	13,467	145	873	705	6,579	6,159	15,610	21,769
	New Hampshire	393	59	452	61	37	98	550	1,094	295	1,389	730	125	3	2	860	2,279	520	2,799	135	170	109	437	2,523	1,127	3,650
	Rhode Island	59	44	103	489	103	592	695	237	1,139	1,375	206	390	16	206	818	562	2,327	2,889	79	157	125	911	766	3,395	4,161
	Vermont	210	189	399	30	63	93	492	752	141	893	305	1	143	19	468	1,599	254	1,853	8	*	195	209	1,802	463	2,265
	Total	2,745	677	3,422	3,752	1,208	4,960	8,382	7,028	8,124	15,152	3,533	1,917	1,228	1,577	8,255	15,211	16,578	31,789	1,441	3,114	1,652	13,087	18,304	32,779	51,083
Middle Atlantic	New Jersey	231	373	604	1,771	1,853	3,624	4,228	2,351	5,317	7,668	36	100	1,309	2,244	3,689	4,300	11,285	15,585	1,391	2,123	4,234	12,240	9,925	25,648	35,573
	New York	3,020	221	3,241	5,424	738	6,162	9,403	8,228	11,581	19,809	1,740	1,130	2,776	1,399	7,045	15,985	20,272	36,257	42	58	7,940	16,434	23,267	36,764	60,031
	Pennsylvania	3,613	1,171	4,784	2,205	894	3,099	7,883	7,789	5,908	13,697	5,516	3,326	46	93	8,981	18,135	12,426	30,561	3,164	4,010	3,996	10,392	25,295	26,828	52,123
	Total	6,864	1,765	8,629	9,400	3,495	12,885	21,514	18,368	22,806	41,174	7,292	4,556	4,131	3,736	19,715	38,420	43,983	82,403	4,597	6,191	16,170	39,066	58,487	89,240	148,427
South Atlantic (North)	Delaware	49	-	49	156	107	263	312	985	706	1,691	319	210	-	-	529	1,353	1,179	2,532	-	-	82	69	1,435	1,248	2,683
	Dist. of Col.	-	-	-	192	155	347	347	-	1,025	1,025	-	-	-	492	492	-	1,864	1,864	-	-	-	779	-	2,643	2,643
	Maryland	788	154	942	1,990	513	2,503	3,445	2,799	2,200	4,999	1,531	825	439	405	3,200	5,711	5,933	11,644	649	80	3,052	2,398	9,412	8,411	17,823
	Virginia	2,523	1,056	3,579	656	643	1,299	4,878	4,514	2,179	6,693	2,993	569	2,041	403	6,006	13,127	4,450	17,577	92	419	1,675	2,878	14,894	7,747	22,641
	West Virginia	308	523	831	78	192	270	1,101	2,030	719	2,749	1,491	115	870	32	2,509	5,222	1,137	6,359	7	24	301	1,160	5,530	2,321	7,851
	Total	3,668	1,733	5,401	3,072	1,610	4,682	10,083	10,328	6,829	17,157	6,334	1,720	3,350	1,332	12,736	25,413	14,563	39,976	748	523	5,110	7,284	31,271	22,370	53,641
South Atlantic (South)	Florida	1,598	1,092	2,690	1,251	799	2,050	4,740	4,990	3,777	8,767	4,293	2,577	206	33	7,109	12,179	8,437	20,616	325	1,463	6,217	4,145	18,721	14,045	32,766
	Georgia	1,630	1,322	2,952	1,332	353	1,685	4,637	5,440	1,798	7,238	2,506	514	1,251	420	4,691	12,149	4,417	16,566	139	359	1,766	5,391	14,054	10,167	24,221
	North Carolina	1,311	1,062	2,373	350	338	688	3,061	4,202	1,698	5,900	8,682	2,109	3	193	10,987	15,260	4,688	19,948	1,845	561	32	2,243	17,137	7,492	24,629
	South Carolina	985	716	1,701	85	159	244	1,945	4,200	1,383	5,583	3,083	440	117	5	3,645	9,101	2,072	11,173	307	926	345	335	9,753	3,333	13,086
	Total	5,524	4,192	9,716	3,018	1,649	4,667	14,383	18,832	8,656	27,488	18,564	5,640	1,577	651	26,432	48,689	19,614	68,303	2,616	3,309	8,360	12,114	59,665	35,037	94,702
East North Central	Illinois	2,365	1,678	4,043	4,094	772	4,866	8,909	8,365	6,928	15,293	1,078	604	2,405	541	4,628	15,891	12,939	28,830	1,472	3,604	2,792	13,160	20,155	29,703	49,858
	Indiana	2,092	1,226	3,318	969	990	1,959	5,277	6,007	2,627	8,634	2,831	623	1,875	711	6,040	14,031	5,920	19,951	120	193	1,133	6,002	15,284	12,115	27,399
	Michigan	2,575	245	2,820	2,540	1,414	3,954	6,774	6,594	5,657	12,251	1,351	417	6,513	992	9,273	17,268	11,030	28,298	37	54	4,415	12,250	21,720	23,334	45,054
	Ohio	3,682	541	4,223	3,353	1,364	4,717	8,940	7,965	5,483	13,448	4,268	1,460	2,320	1,959	10,007	18,776	13,619	32,395	101	427	4,725	12,074	23,602	26,120	49,722
	Wisconsin	1,346	132	1,478	541	9	550	2,028	5,400	2,230	7,630	1,685	570	1,510	915	4,680	10,073	4,265	14,338	41	52	1,155	5,335	11,269	9,652	20,921
	Total	12,060	3,822	15,882	11,497	4,549	16,046	31,928	34,331	22,925	57,256	11,213	3,674	14,623	5,118	34,628	76,039	47,773	123,812	1,771	4,340	14,220	48,821	92,030	100,934	192,964
West North Central	Iowa	1,346	211	1,557	203	84	287	1,844	4,944	1,362	6,306	-	-	1,522	313	1,835	8,023	1,962	9,985	80	51	886	2,471	8,989	4,484	13,473
	Kansas	810	215	1,026	310	78	388	1,414	3,760	1,045	4,805	574	23	1,417	410	2,424	6,777	1,866	8,643	86	64	1,080	2,442	7,943	4,372	12,315
	Minnesota	373	566	939	751	800	1,551	2,490	5,216	2,064	7,280	1,083	33	2,556	161	3,833	9,794	3,809	13,603	24	114	1,273	3,647	11,091	7,570	18,661
	Missouri	1,926	963	2,889	1,423	696	2,119	5,008	5,379	2,017	7,396	2,281	387	17	36	2,723	10,568	4,559	15,127	126	642	1,848	6,034	12,542	11,235	23,777
	Nebraska	775	185	960	60	45	105	1,065	2,852	628	3,480	635	32	535	71	1,381	4,893	943	5,836	4	-	1,002	1,865	5,899	2,808	8,707
	North Dakota	334	118	452	11	22	33	485	1,103	147	1,250	135	4	430	13	643	2,181	197	2,378	1	2	662	401	2,844	600	3,444
	South Dakota	[illegible]	[illegible]	[illegible]	19	[illegible]	[illegible]	[illegible]	[illegible]	145	[illegible]	258	31	379	25	693	2,862	279	3,141	22	7	576	452	3,460	738	4,198
	Total	[illegible]	2,524	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7,452	32,285	5,023	517	[illegible]	1,030	13,532	45,139	13,515	58,[illegible]	343	990	[illegible]	17,312	52,968	31,[illegible]07	84,[illegible]
East South Central	Alabama	[illegible]	[illegible]	[illegible]	11	[illegible]	94	[illegible]	[illegible]	1,515	5,40[illegible]	1,335	55[illegible]	1,351	104	3,14[illegible]	8,186	2,409	11,095	41	50	753	3,083	9,980	5,042	15,022
	Kentucky	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,320	5,180	3,410	480	215	32	4,137	9,255	2,752	12,007	1,010	630	1,095	1,884	11,360	5,266	16,626
	Mississippi	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	99	[illegible]	[illegible]	552	4,24[illegible]	803	82	220	182	2,037	6,357	1,504	7,861	18	13	535	1,293	7,011	2,810	9,821
	Tennessee	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,504	[illegible]	4,755	1,376	6,141	124	159	554	47	1,734	9,339	3,796	12,134	45	4	1,798	4,021	10,181	7,821	18,002
	Total	3,491	[illegible]	[illegible]	1,74[illegible]	[illegible]	[illegible]	10,403	15,01[illegible]	7,554	21,577	[illegible],472	290	3,200	455	11,117	32,135	10,961	43,097	1,114	597	4,282	10,281	37,532	21,339	59,471
West South Central	Arkansas	[illegible]	[illegible]	[illegible]	[illegible]	8	[illegible]	1,331	2,[illegible]	[illegible]	3,51[illegible]	2,075	318	250	37	2,680	5,343	1,503	7,546	58	58	666	1,374	5,667	3,045	9,712
	Louisiana	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,041	[illegible]	2,[illegible]54	1,34[illegible]	3,9[illegible]4	3,412	542	11	1	3,966	7,774	2,924	10,698	616	375	661	2,289	9,051	5,588	14,639
	Oklahoma	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4,591	1,402	[illegible]	[illegible]	292	712	247	2,219	7,706	2,855	10,561	208	113	975	3,282	9,889	6,250	15,139
	Texas	[illegible]	[illegible]	4,504	[illegible]	1,[illegible]65	[illegible]	12,[illegible]25	11,311	[illegible]	18,239	7,525	2,074	-		9,705	23,551	17,008	40,559	1,250	920	2,473	13,790	27,294	31,718	59,002
	Total	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21,442	[illegible]	31,[illegible]64	14,041	3,221	983	285	18,580	44,984	24,390	69,374	2,132	1,475	4,775	20,735	51,891	46,501	98,492
Mountain	Arizona	[illegible]	1,254	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,1[illegible]4	[illegible]	1,45[illegible]	40[illegible]	48	4[illegible]3	1,899	2,219	4,353	2,770	7,123	185	15	373	1,519	4,911	4,304	9,215
	Colorado	[illegible]	15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	144		-	1,061	4,330	2,365	6,695	19	48	[illegible]	2,609	5,621	5,022	10,643
	Idaho	[illegible]	504	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,141	203	1,344	325	24	303	52	704	2,520	374	2,894	3	2	671	525	3,194	902	4,096
	Montana	[illegible]	55	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,581	400	53		-	453	2,558	328	2,886			734	460	3,292	788	4,080
	Nevada	[illegible]	13	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	367	255	53	35	155	519	1,708	562	2,270	22	20	100	650	1,830	1,232	3,062
	New Mexico	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,924	903	129	5	*	938	3,546	996	4,542	72	437	396	574	4,014	2,107	6,121
	Utah	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,2[illegible]	537	438	98	132	1,205	2,553	1,517	4,070	35	125	403	625	2,992	2,268	5,260
	Wyoming	44	421	[illegible]	13	[illegible]	34	[illegible]	913	125	739	210		13	9	239	1,903	[illegible]	1,279	2	3	275	324	2,080	502	2,582
	Total	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3,447	[illegible]	[illegible]	405	[illegible]	2,257	8,098	23,371	9,087	32,458	339	551	4,224	7,387	27,934	17,125	45,059
Pacific	California	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	944	3,547	3,729	10,547	24,573	33,195	[illegible]	[illegible]	[illegible]	7,690	36,479	32,847	[illegible]	103,818
	Oregon	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,333	3,534	1,034	293	723	456	2,496	5,454	2,795	8,259	36	14	1,370	1,996	5,870	4,595	11,565
	Washington	[illegible]	[illegible]	1,454	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3,530	[illegible]	521	1,597	342	3,489	6,207	4,646	10,853	15	144	2,826	3,902	9,049	8,692	17,741
	Total	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	32,[illegible]	[illegible]	1,648	5,[illegible]	5,137	17,132	36,344	40,536	[illegible]	545	1,355	11,875	42,257	48,955	84,[illegible]58	133,124
[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	213,[illegible]	[illegible]	[illegible]	24,784	42,925	21,639	170,225	385,805	241,100	[illegible]	15,747	22,536	[illegible]	218,354	479,548	[illegible]	[illegible]
	Alaska			-				-	205	101	406	105	35	-	-	141	411	136	547			107	212	518	355	873
	Hawaii			[illegible]				[illegible]	[illegible]	402	[illegible]	[illegible]	63		-	296	830	815	1,645	-	-	404	572	1,234	1,48[illegible]	2,[illegible]21
[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	214,034	[illegible]	[illegible]	24,[illegible]	42,925	21,639	170,662	387,046	242,051	629,097	15,747	22,536	78,507	219,345	481,300	483,[illegible]	[illegible]

1/ Traveled way [illegible] in the Interstate System status of completion group 2. This group consists of mileage which is adequate for present traffic but will need further construction and improvement [illegible] Interstate standards. In some instances these are divided highways with full control of access and in others they are two, three or four-lane undivided highways with no access control.

* [illegible]

[illegible] and [illegible]

tered and 19 percent of all travel. Similar figures for buses were less than 1 percent.

Average performance for all vehicles in 1967 differed somewhat from that reported in 1966. The average motor vehicle traveled 9,755 miles in 1967, half of it in cities, and consumed 785 gallons of fuel at a rate of 12.42 miles per gallon. The average passenger car traveled 9,582 miles and consumed 686 gallons of fuel at a rate of 13.96 miles per gallon.

According to the State estimates, the traveled way of the Interstate System carried 167.7 billion vehicle-miles, or 17.4 percent of the total 1967 travel on all roads and streets. The traveled way consisted of 22,288 miles of Interstate System highways now in use and of 18,712 miles of existing connecting highways. Service for this total mileage will be provided by the Interstate System when it is completed. From the State estimates it is expected that by 1975 the 41,000 mile Interstate Systen comprising little more than 1 percent of th total road and street mileage of the Unite States, will carry more than 20 percent of th total 1,213 billion miles of travel estimate for 1975.

According to the State estimates of 196 travel, all Federal-aid systems combinec which includes about 25 percent of all road and streets, carried 65 percent of all travel.

NEW PUBLICATIONS

The Bureau of Public Roads has recently published two documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Fatal and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967

Fatal and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967 (45 cents a copy), is a 36-page publication and is the first in a contemplated annual series to present accident, fatality and injury rates per 100 million vehicle-miles by State and administrative highway system. Included also are similar rates based on numbers of registered vehicles, licensed drivers, and population. In addition to the rates, the actual numbers of highway miles in service, vehicle miles, accidents, fatalities, and injuries are shown in supporting tables. This compilation is based on reports submitted by the State highway departments.

Standard Plans for Highway Bridges, Volume IV, Typical Continuous Bridges, 1969

Standard Plans for Highway Bridges, Volume IV, Typical Continuous Bridges, 1969 ($1.50 a copy), is a revision of the 1962 edition with respect to bridge widths and current design specifications. These plans are intended to serve as a useful guide in the development of suitable and economical bridge designs. An effort has been made to give sufficient information on all plans so that they may be readil modified in the preparation of contract drawings.

The volume contains four sets of plans fo substructure and superstructure of four-span continuous bridges, including a concrete voided slab, a concrete T–beam, a concrete box girder, and a composite welded steel girder.

A variety of bent and footing types have been detailed for the four bridges. Intermediate supports include monolithic bents, framed bents, and solid wall piers. Footing types include pedestal piles, steel bearing piles, friction piles, and spread footings.

Bridge roadway width is 44 feet with HS 20–44 live loading for the standard 2-lane, two-directional roadway.

HIGHWAY RESEARCH AND DEVELOPMENT REPORTS AVAILABLE FROM CLEARINGHOUSE FOR FEDERAL SCIENTIFIC AND TECHNICAL INFORMATION

As the results of research are useful only when they reach those who can implement them or apply the knowledge gained to other endeavors, the problem of how to make research reports available to interested persons has always been a concern of administrators. This problem was answered, at least in part, by the establishment of the Clearinghouse for Federal Scientific and Technical Information, a repository for technical documents organized in 1964 on the foundations of the Department of Commerce's Office of Technical Services.

The Clearinghouse collects research reports that are the results of work performed by Government laboratories or by industrial firms and private institutions under contract to sponsoring Federal agencies. More than 50 departments and agencies supply 50,000 reports to the Clearinghouse each year. The Clearinghouse announces, reproduces, and sells the reports at a nominal cost. Many organizations use them to produce new items for management, improve production processes, reduce costs, solve technical problems, prepare bids on Government contracts, or keep abreast of the state-of-the-art.

Among the documents collected by the Clearinghouse are the published results of federally supported highway research and development projects. As with all the documents on file, these publications are available as either microfiche or paper copies. Microfiche is a 4- by 6-inch sheet of film that contains as many as 70 pages of a document. Paper copy is produced by offset printing. Microfiche copies are more economical than paper copies, are easier to handle, and can be filed readily.

As a service to the readers of PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, listings of these highway research and development publications available from the Clearinghouse will be published, as space permits, in future issues. These publications will be listed chronologically beginning with the first report collected by the Clearinghouse and will continue until all the highway research and development publications on file at the Clearinghouse have been listed. From that point, each issue will list the reports that will have been collected by the Clearinghouse since the last issue. Each listing will include the accession (stock number) of each report. Any highway research and development publication can be purchased by sending the stock number and a check or money order to: Clearinghouse for Federal Scientific and Technical Information, Sills Building, 5285 Port Royal Road, Springfield, Va. 22151. Paper copies are priced at $3 each and microfiche copies at 60 cents each. The Clearinghouse requires prepayment on all orders.

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A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–34 are available upon request addressed to Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. 20591.*

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Third Generation Destination Signing—An Electronic Route Guidance System

BY THE OFFICE OF
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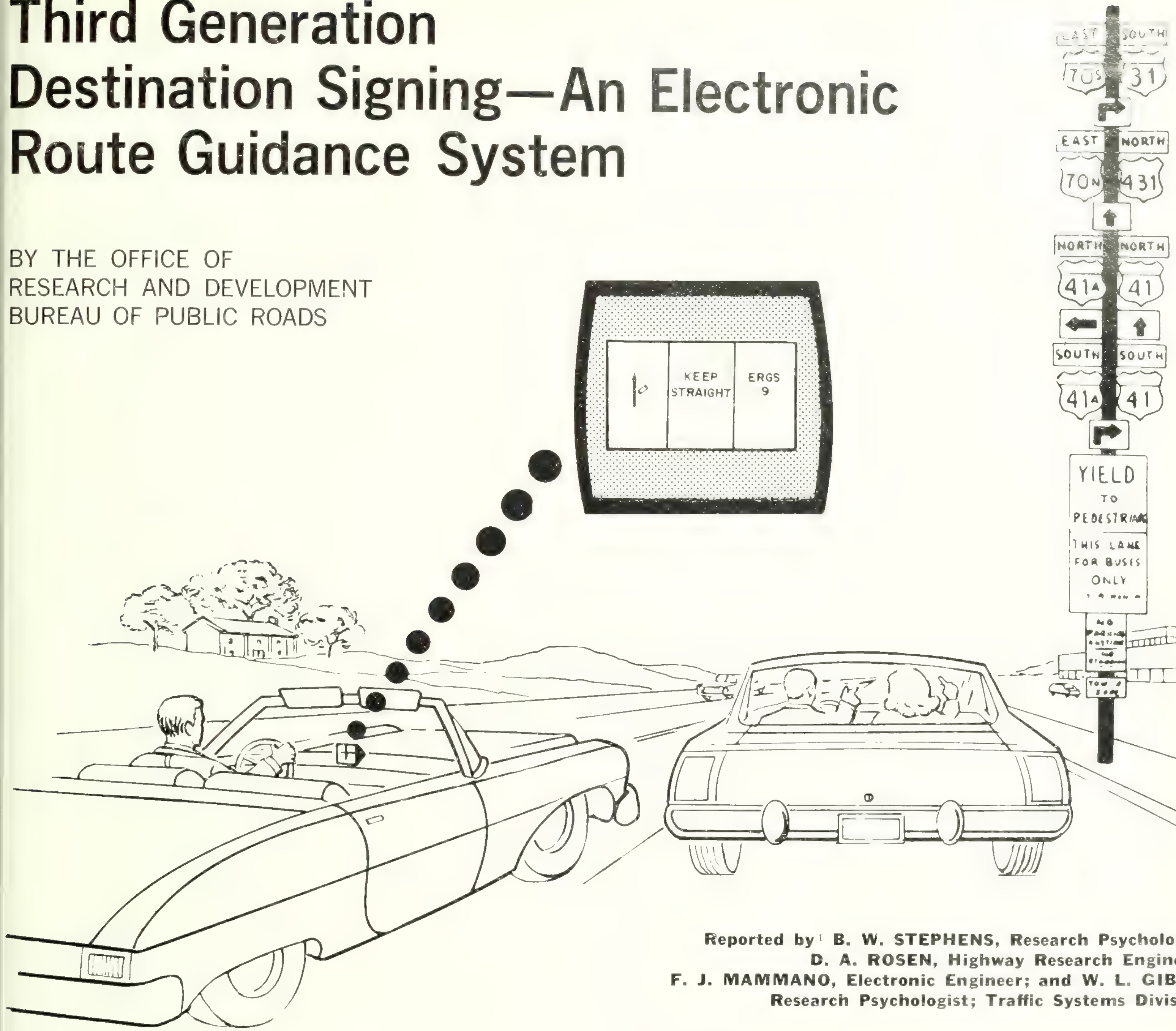

**Reported by[1] B. W. STEPHENS, Research Psychologist
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The problem of routing an automobile driver safely and efficiently from his origin to his destination is analyzed and present highway routing and navigational methods are examined in the framework of a systems analysis of the highway routing subsystem. In this context, the functions, characteristics, normative operation, and constraints of the present routing subsystem are delineated. The analysis traces the implications of removing three major constraints from the present highway routing subsystem: Open loop communications with drivers, utilitarian rather than individualized communication, and treatment of the highway and the automobile as separate entities.

Techniques of communication that can remove system constraints are considered, and a route-guidance-system concept and a plan for implementing it are presented. The implementation plan for the concept, consisting of a multiphased research and development program currently underway, includes studies that will lead to the design of an electronic route guidance system, a prototype of which will be installed on an actual highway network to be thoroughly tested and evaluated.

If all highway route marking signs of the prefreeway era are considered first generation and those of the freeway era, second generation, then the new concept and routing system described here can be properly termed third generation destination signing.

Introduction

IN this article, highway routing, or navigational methods and procedures, as they exist today are analyzed; the functions, characteristics, normative operation, and constraints of routing methods are delineated; and several novel techniques for communicating directional information to drivers are examined. A research and development effort to transform into an operational system a concept of a route guidance system is also examined. This system concept, which is now being implemented, is the outgrowth of synthesis of certain techniques combined with a fundamental analysis of routing. Considering all prefreeway route marking signs as first generation, and freeway signs of today as second generation, the assembly of software and hardware components of the newly conceived system can be viewed as a third generation destination signing system.

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[1] Presented at the 48th annual meeting of the Highway Research Board, Washington, D.C., January 1969.

During the current decade it has become fashionable to conduct analyses of existing man-machine systems to determine whether they can be made more efficient or whether man should be supplanted by mechanical or electronic functional analogs. Usually there are severe symptoms of disorder before the analyses are conducted or the solutions that have been developed are implemented.

Symptoms of disorder seem to exist for vehicular traffic operations. Many individuals are keenly aware of the degree of system disorder, including the infrequent traveler who journeys to major metropolitan centers, the traffic engineer who struggles from day-to-day with the movement of vehicles in and out of such areas, the weekend traveler who frequents his favorite recreation areas, and those who have observed aerially the minute-to-minute variations of traffic density. The evolution of highway transportation has been so gradual that the nation's more than 100 million drivers have adapted to the disorders with only an occasional discouraging word.

Such adaptation should not be unexpected, as the present highway transportation system and its associated methods of construction and control are the products of a gradual evolution dating back to the Roman Empire's Appian Way. Methods of routing and control for English and American personal transportation in the 19th century was astonishingly similar to present methods. During the mid-19th century in London, safety officers performed the functions of adaptive signals at high volume intersections, and in 1850, traffic congestion in New York City was often dissipated by the policeman's night stick.

About 1920, the motor vehicle became a practical means of extending the environs of most Americans, and the problem of finding one's way became a reality for many drivers. Signing devised by the automobile clubs and local governments became a solution, or rather, a myriad of solutions for route direction. Oil companies and automobile clubs served their customers and members by providing maps and road signs of different complexity. Various inventions, such as Jone's moving map, also provided guidance to drivers. Although the errors in planning and guidance were frequent, the penalties of the errors usually were minor.

Today, errors inflict penalties that are considerably greater, and only recently have drivers become aware of the costs resulting from congestion and from difficulty in finding one's way. Although the costs were intuitively perceived, they were seldom measured, and when they were measured, greatly simplified assumptions were made. Frequently, solutions to a tremendously complex set of problems have been seized upon without a knowledge of how to measure the efficiency of such solutions.

There is no attempt here to provide a solution to traffic congestion problems or to [illegible] movement. What is attempted is the tracing of the implications of removing the following three major constraints from the highway transportation system:

- Communication with drivers primarily exists as an *open loop*.
- Communication cannot be individualized but must be utilitarian.
- The highway and the automobile are separate entities.

The difficulties and benefits associated with removal of each of these constraints will be delineated in the following systems analyses. The most striking benefit of removing them, added to *real time* measurement of traffic proficiency, is the possibility of distributing vehicular traffic much more uniformly than is possible with the present system of routing. This strategy, which is mutually advantageous both to individual drivers and roadway authorities, has been labeled *route control* by Gazis (*1*).[2]

A Systems Engineering Analysis of Route Guidance

The specific system to be analyzed is a part of the highway transportation system. It is a collection of techniques, methods, procedures, and devices that will be referred to as the routing subsystem.

Although systems engineering activities will not be described here, some important steps will be delineated. Examples of several excellent books that serve as an outline for this endeavor are those by Chestnut (*2*); Flagle et al. (*3*); and Goode and Machol (*4*). Questions that will be addressed in this analysis, and which will serve as an outline for analyzing the routing problem, are as follows:

- What are the functions of the system?
- How can we characterize the operating environment of the system?
- What information do we possess about the system's current operation, and how should it operate?
- What constraints on system design exist?
- What tradeoffs exist between different proposed system solutions?

The functions of the highway routing system are the provision of *course and routing* information that is available to the driver at appropriate roadway nodes, and the provision of error signals when an improper course is selected or of other adaptive controls when improper maneuvers are made at junctures. These should not be considered independent functions as will be shown later for the proposed design.

The functions can be further delineated by specifying the operations, or tasks, that are inherent in trip generation—trip planning, path control, choice point path selection and control, and terminal or destination recognition. Each of these tasks, demanded of the driver at present, becomes more important as the trip becomes longer, as congestion is more variable, and as the perceptual and cognitive capabilities of the driver become low c stressed.

At present, trip planning requirements ar served by human memory, previous transfe able experience, and the ability of the drive to make accurate decisions on relative di tances and durations of different routes. Di tance and duration are derived primarily fro maps that also can serve to prepare the drive for certain static elements of the trip. Othe information on construction activity or con gestion on various links of the trip usually i not reliably available to the driver during th planning stages of *trip making*.

In congested areas of the trip, frequen changes in routing may be necessitated b different levels of traffic demand. The drive who is very familiar with the highway net work that he is traversing has comparativel little difficulty in selecting alternate route But even in such an environment, he is face with situations in which his memory may b taxed, his decision process rate exceeded, o information on the state of congestion o alternate routes simply is not available. H frequently becomes what has been labeled *local stranger*.

Path control primarily involves perceptua information processing and psychomotor control tasks that are related to judgments abou overtaking, following, steering, accepting gap between vehicles in the traffic stream, an passing other vehicles. The degree of inter action between uncertain routing decisions an vehicular control still remains to be resolve empirically, although some data have bee obtained. It has been suggested in Brown and Poulton's analysis (*5*) of spare capacity unde various conditions of traffic load and intersection frequency and in Sender's technique (*6*) of calibrating information load of driver that traffic density and highway geometry have a strong influence on the variability o psychomotor control.

More to the point is a simulation study o time sharing by Stephens and Michaels (*7*). This study, although conducted in the laboratory, indicated that the proficiency of a simpl tracking task similar to steering was influenced by the amount of signed information along the roadway and by the subject's expectancy for such information.

Field studies that directly measure speed and lateral variability as a function of the amount of destination information to be *stored in the human memory* are now being conducted by researchers of the Bureau of Public Roads.

Further, environmental uncertainty will aggravate increased stress or decreased alertness brought about by alcohol, fatigue, or other causes. It is remarkable that accident rates, which are compounded by destination information that is preserved in the driver's imperfect memory and by his difficulties in locating and recognizing signed information, are not higher than those that have been currently reported.

Proficiency of executing tasks associated with driving differs among drivers. Persons who have a great deal of difficulty in processing information about the highway environment pace these tasks by decreasing driving speed, which consequently, increases the variability of relative velocities of vehicles on the highway (*6*). Empirical relations between vehicular relative velocity and high accident rates for rural highways have been established by Solomon (*8*). Speed differences as low as −10 m.p.h. were reported to perpetrate about six times the accident involvement rates as situations in which the vehicles did not vary from the average speed of traffic. Recently Cirillo (*9*) reached essentially the same conclusions on the Interstate Highway System, although the results were more dramatic. An interesting point about this relationship between speed deviation and accident involvement rate is that it is not symmetrical about the mean speed of traffic; the minimum involvement rate occurs when vehicles operate at speeds somewhat above the mean speed of traffic.

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[2] Italic numbers in parentheses identify the references listed on page 200.

Choice-point path selection and vehicular control in the vicinity of highway junctures also seem to be strongly affected by the driver's difficulties with information processing, decisionmaking, and psychomotor response changes. Mullins and Keese (*10*) indicated that for freeway ramps, 0.72 accidents per million vehicle miles (APMVM) occurred at off-ramps, and 3.91 APMVM at on-ramps. The converse was reported for California roadways by Lundy (*11*) who indicated that off-ramp accident rates vary from 0.62 to 2.19 APMVM, and on-ramps from 0.40–0.93 APMVM. Different geometrics, signing, and traffic volumes probably accounted for these differences.

According to Cirillo, a rapid escalation in accident rates occurs as one approaches the decision point at interchanges. From 2 to 4 miles before the juncture, on urban Interstate highways, the accident rate is approximately half the rate in areas between the gore and 0.2 mile immediately preceding it. Comparable rural Interstate sections yield only about an increase of ½ as one moves closer to the highway juncture.

Covault et al. (*12*) attempted to determine the effects of lateral dispersion and speed changes in the vicinity of interchanges as a function of the redundant destination information provided by both audio and visual signs. Because of the redundancy of presentation, the probability of directional information reaching drivers increased, and the lateral stability and speed constancy also increased.

As the driver finally approaches his destination, how does he know he has arrived? There is a rapid sequence of decisions associated with locating the specific goal sought by the driver. He must park, usually to minimize the walking distance from his vehicle. In parking lots, on city streets, and in the vicinity of shopping areas, the demands upon judgment are many. Subjective estimates of the distances involved during location and parking maneuvering are markedly poor, probably owing, in part, to the fact that the driver infrequently has to make these judgments. There is a cascading of decisions that both demands rapid judgments and consumes a substantial part of most trips. In current studies at the Bureau of Public Roads, the scaling accuracy and judgments associated with turning maneuvers and distance judgments, such as those required in parking lots, are being dealt with.

As approximately 60 percent of trips are 5 miles long, or less (*13*), final selection of a specific parking spot can occupy from ½ to 2 minutes—a considerable portion of the travelers time. It has been estimated that from 2 to 24 percent of travel time is devoted to parking, and for the driver whose trips are almost exclusively less than 5 miles long, the parking time approaches 40 percent. It is not implied that the terminal phase of travel is constrained by routing information alone; parking availability is undoubtedly much more important in reducing the terminal part of travel time.

For the driver who is unfamiliar with the characteristics of the terminal, the problem is much more critical. The driver must search for cues, that may or may not be prominent, to locate his destination and these cues should indeed tell him when he has arrived. However, there is a period of intervening activity between assimilation of the cues and turning off the motor that requires an empirical analysis, which is planned by Public Roads researchers who will use the information-calibration technique devised by Senders (*6*).

The effectiveness of fulfilling the tasks that compose the highway routing subsystem depends largely on the specific characteristics of the environment in which drivers negotiate their trips.

Characteristics of the operating environment

The environment in which individual trips are generated is characterized by roadway segments that frequently consist of multiple parallel paths connected at junctures directly or by intervening roadway segments. For traffic assignment, this complex structure comprises the simpler notions of roadway segments (links) and intersections (nodes). When treated this way, a variety of techniques can be applied to solve shortest-route problems (*14*). Unfortunately, historical data on averages of travel patterns has little applicability to demands on particular roadways, except possibly at times when operation is near saturation.

A much more microscopic analysis of trip distribution and the highway environment would consider at least the following roadway-related characteristics to successfully model the transition of vehicles from one set of roadway segments to another, or from the origins of the trips to the respective destinations:

- Highway lane during the vehicle's approach to each roadway junction.
- Conspicuousness of the junction including signing, roadway delineation, and highway alinement, as a function of distance from the junction.
- Geometric channeling near the juncture.
- Signaling and other control techniques.

The extent to which each of these characteristics facilitates or inhibits the flow of traffic depends largely on the following modulation characteristics:

- Traffic—the density and flow characteristics.
- Pedestrian flows.
- Weather restrictions.
- Vehicle handling characteristics.
- Driver perceptual sensitivities and response capabilities.

Several microscopic models, incorporating various combinations of parameters, already have been developed (*15, 16, 17*). To say that microscopic analysis of intersection and interchange operation is required is not to say that network operation and corridor operation analyses are not required.

As pointed out earlier, the objective of route control is to distribute vehicles on the roadway network and increase traffic flow throughout the network. To achieve this objective it is necessary to ascertain the diversity of trip origins and destinations, the time distribution of departures and arrivals, and the existence of parallel routes in different corridors. A discussion of anomolies of traffic operation caused by various stressors is beyond the scope of this article; it suffices to say that inclement weather, poor vehicular acceleration characteristics, high vehicular density, and driver capabilities all can influence traffic operation to a considerable degree. Perhaps a more salient question is, "How should traffic operate on the highway network?"

Normative system operation

In the preceding discussion, many characteristics of the traffic environment, as they affect distribution of vehicles on the network, were considered. It is now necessary to define the highway system in general operational terms to subsequently consider the effects of certain constraints on the traffic environment.

Perhaps the most general characteristic of total highway-system operation is the degree of entropy that the system has. Entropy in the context here can be viewed as the variation of traffic flow on the totality of all roadways, or links, within the system. Hence *Traffic Systems Entropy* can be defined as the sum of the variances of traffic flow which, obviously, is conditional and depends on the time period over which measures of flow are gathered. Accordingly, reference can be made to yearly, monthly, daily, *rush hour*, or entropy for any time period, depending on the purpose of the inquiry. For example, if it were necessary to determine whether several signal strategies for different daily time periods were warranted, flows would be measured on a base of one hour or less.

Of the several advantages to defining entropy in information-theoretical terms, the major advantage is the partitioning of information without regard to metrics (but not without regard to logic). The same measure of information can be arrived at in several ways, including the speed variability of pairs of vehicles, of individual vehicles, etc. Shannon's classic on information theory (*18*) has shown that, logically, variance can be transformed into information (or uncertainty or entropy).[3] Using the same types of formulation, throughput can be calculated for individual intersections or extended to broader networks. Other formulations, including conditional flow, can also be developed.

It is beyond the scope of this article to demonstrate analytically the relations between different microscopic measures of information for vehicular traffic flow. It suffices to state that once the transformations between flow and vehicular-speed and acceleration patterns are made, different equivalent operations should provide equivalent results, although some estimation processes are more efficient than others.

Treated in such a context, the capacity of a highway arterial or network section can be considered as a maximal-transmission or bandwidth problem, requiring decision-rules for recommending alternate routes when flow approaches theoretical capacity. Nearly all improved routing techniques are developed to increase the existing level of service so that it approaches theoretical capacity.

The proposed method of routing drivers through highway networks has been based on a partial analysis of flow data, as sufficient data exists to effect detailed analytic solutions. Network efficiency, as it is now determined, is a macroscopic measure that does not treat minute variations in the system. It is meaningful to planners, designers, and officials who are operating a traffic control system, but not to the individual driver who is primarily concerned with completing his trip quickly and reliably (*19*). As long as there is a monotonic relation between these two criteria, no difficulty exists. Otherwise reconciling the two becomes an optimization problem.

An important step in the development of the proposed routing system was a study by Carter et al.,[4] in which an attempt was made to develop a conceptual scheme for measuring the effectiveness of highway networks. Their approach was reductionistic and strongly oriented toward driver benefits rather than to total network advantages. The following excerpt from this unpublished report illustrates hypothetical relations between certain of these measures:

"The criteria for effective operation of a highway network must be operationally defined in terms of level of service of the network, safety, and comfort, and convenience to the operators. Values must accordingly be assigned to each of these criteria and finally alternate systems evaluated in such terms. One of the most difficult problems associated with such evaluation is how to optimize among criteria measured in differing terms. The evident answer lies in devising a common measure, or metric, to ascertain whether certain criteria which seem really important to the planner are actually important to the user. For example, a number of researches have indicated that drivers appear to choose routes which provide time savings even though drivers might have to drive much greater distances. Hence it would appear reasonable to employ time savings in place of or in some weighted combination with physical distance over the network as a criterion of network performance, at least for a number of types of travel.

"Michaels (*20*) has suggested an even more comprehensive formulation which provides a common index of subnetwork usage. This measure incorporates branch distance (in miles), relative distances on high-design facilities and low-design facilities and average travel speeds and distances. An equivalency measure can be established between these values and the level of stress impinging upon the operator. This provides for a scaling method relating certain of the variables associated with comfort and convenience.

"For various levels of this measure, both the effect upon traffic operations (primarily in-steam turbulence and turbulence at junctures or nodes) and upon safety (the probability of collisions weighted for severity) must be determined. . . .

"The level of traffic or vehicular performance can be operationally dealt with at a molecular level. Turbulence has been operationally defined for the branch situation by a number of investigators. 'Acceleration Noise' (standard deviation of acceleration) has been employed for the in-stream case by several investigators. While this measure is gross it is becoming widely employed to differentiate various levels of traffic operation. Roeca (*21*) has developed a more comprehensive analytic technique for evaluating the effects of particular disturbances introduced into the traffic stream (e.g., the effects of a stopped vehicle on the road shoulder upon in-stream speed variations). . . .

"Molecular operation of vehicles at nodes has also been explicitly considered by Bureau of Public Roads personnel while at least two contractors, Covault (*12*), Mace, et al. (*22*), have explicitly developed criteria for effective juncture operation. These criteria include operator-vehicular performance in the traffic stream prior to the diverging operation as well as performance on the ramp itself."

Although there is little assurance that the measures presented here are relevant, the absence of data necessitates, as a guide for further research, the development of certain hypothetical relations between driver efficiency and network efficiency. If driver benefit is considered the percentage of drivers taking the shortest temporal routes from origins to destinations, it is more meaningful to talk about the impedance of the highway system, I(S), because of the difficulty of prescribing upper bounds for obtainable speeds on different highway segments.

Network impedance can be taken as the difference between highway capacity, or maximum flow, and the actual flow on each highway link summed over all the links in the network. If an upper speed bound, V(S), could be prescribed, then system efficiency, E(S), might be taken as that speed less I(S) or simply $E(S) = V(S) - I(S)$. Although the scales are mixed—one ordinal, the other ratio—the relations between network impedance, as defined here, and the percentage of drivers taking the shortest routes are shown in figure 1.

It is suggested by these relations that, when *free flow* conditions are maintained, *familiar* drivers are not obliged to change or reduce their velocities substantially; they will traverse a route because of its intrinsic benefits. The route is simply a preferred one. As capacity is approached on a particular link, average speed decreases, and the driver *scans memory*, receives radio communication about traffic conditions, or employs passenger knowledge and preference or maps to choose alternate routing *rationally* and improve his route selection. The *unfamiliar driver* who has not previously traversed the route has little to facilitate his travel. Hence, there seems to be a monotonically increasing function between driver benefit and network efficiency for *unfamiliar drivers,* but network efficiency would be less than maximal when *unfamiliar drivers* choose shortest temporal routes.

At least one more concept should be introduced into a generic analysis of network operation: the notion of the degree of adaptation afforded by the system dynamics. Traditionally this concept contributed little to the development of routing systems, although within the last few years, many dynamic operations have been reported in connection with traffic operations—freeway surveillance and control systems (*23*), helicopter communication to drivers (*24*), and adaptive signalization techniques (*25*).

It is expedient to develop some notions of relative benefits when the system is treated either in a static context, with infrequent updating—during a.m. or p.m. rush hours—or in a dynamic context with *real time* data provided for updating *best route* solutions. A routing system in which instructions to drivers are based on *best route* solutions derived from

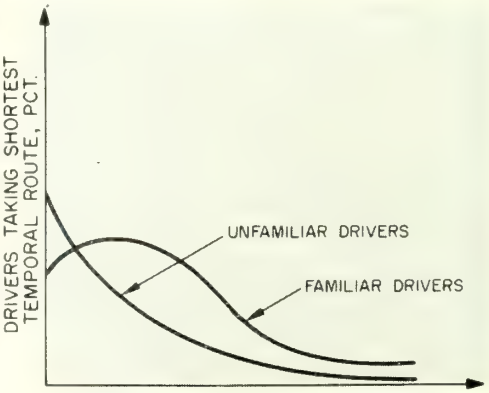

Figure 1.—Relation between driver benefits and highway system impedance.

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[3] [illegible] over a fixed time period for [illegible] Maximum [illegible] could [illegible] be [illegible] technique [illegible] will [illegible] however [illegible] probably [illegible]

[4] [illegible] *Highway [illegible] and [illegible]* [illegible] rter, R. E. Emery, B. W. [illegible] Public Ro[illegible]

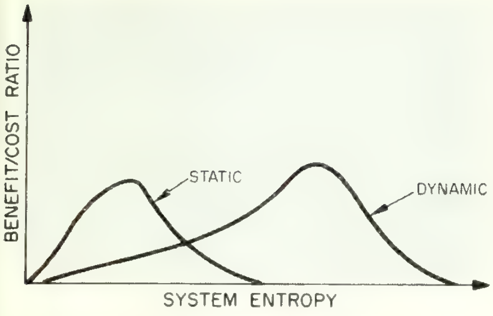

Figure 2.—Roadway system value relations.

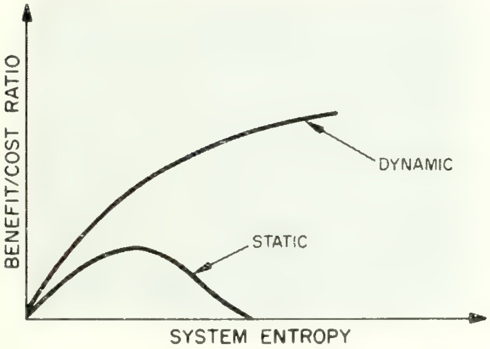

Figure 3.—Individual value relations.

historical data will be called a *static routing system*. A routing system in which instructions to drivers are based on data supplied by air and ground observers and by other surveillance techniques, with solutions updated frequently, will be called a *dynamic routing system*. At present there is little reason to provide routing information, as it could be used only for signalization schemes. But if it is assumed that routing information could be used to increase the throughput at highway junctures, as well as to direct drivers, its provision attains practical significance apart from theoretical significance.

One approach to the problem of providing a suitable system is to map system entropy into a measure of a benefit-cost ratio for each type of system. Total system entropy must be established, or evaluated for subnetworks. Obviously each benefit and cost must be made operational and reflect an extrapolation for some period beyond the initial installation. Benefits are assumed to be weighted cost-functions or another common metric, such as that discussed in Michaels' article (*20*). Such a scale is of little value unless it is either the interval type or the ratio type.

A number of tradeoffs in selection of benefits must be made. In the transportation of people, the *bread and butter* is the level of service afforded by the different highway sections. Construction of additional freeway lanes is an expensive proposition with costs in some urban areas as much as several million dollars per mile. Deployment of guidance aids usually is assumed to develop linearly with system entropy, although stepwise implementation seems to be a better extrapolation of the historical trend.

To highway officials, the salient benefits from any guidance system would be safe and efficient operation of the highway plant translated into reduced construction costs, fewer hazardous appurtenances, and lowered operating and maintenance costs. At present, when levels of service fall below established criteria, costs usually increase, as highway lanes, interchanges, parallel roadways and signalization systems are added to the system. To the driver, the salient benefits of a guidance system would be stopped delays, decreased travel time, lowered fuel costs, and less stress.

A cumulative logarithm function for benefits has been assumed and is expressed as:

$$B = a\left(1-\int_0^x e^{-gx}dx\right)$$

Where,

a, g are weighting values

x is the number of aiding units

B is the measure of benefit

The costs have been assumed to be of the same form, except that the weighting constant associated with the exponent is larger—becomes asymptotic more rapidly. The limit, then, of benefit-cost (B/C) ratio is a constant. When the B/C ratio is compared to system entropy, returns begin to fall off beyond some maximum value.

But such routing strategies do not permit responsiveness to changes in traffic demand, unless routing is altered according to historical data or operation is responsive to the minute-to-minute alterations in demand. There is no obligation to be responsive to low-flow, high-concentration conditions, because such persistent, obvious, bottleneck conditions could be alleviated by new construction.

The initial costs of a dynamic system undoubtedly would be high, but there would be a decrease in cost as engineering production increased, although the decrease would be less dramatic. The benefits for a low entropy system should be about the same as those for a static system, but as entropy and impedance increase, the benefits should increase as a logarithmic relation. These conditions suggest more gradual increase toward a maximum when the B/C ratio is taken relative to system entropy. These system and individual-value relations are plotted in figures 2 and 3.

Benefits to individual drivers mainly continue to increase as a system of routing aids are implemented unless they actually have an adverse effect on travel. For the individual driver, direct costs are fixed and indirect costs are nominal. Functions for static and dynamic systems are inverted-U shapes; however as system entropy increases rapid obsolescence would be expected. Only in areas with persistently small populations would a truly static system be expected to be useful.

Maximum B/C ratio for a programed routing system undoubtedly lies somewhere in between the two prototypes described here. A truly static system will not have practical utility unless it would be applicable to only one time period—non-rushhour—or it would be employed only in areas where prohibitions do not change for daily time periods. Subsequent discussion deals only with the programed-type or dynamic system.

System constraints

Historically there have been many constraints on methods to provide good-quality, reliable highway navigational aids. These constraints might be loosely classified as either technological or socio-economic. While many socio-economic constraints markedly affect system effectiveness, they will not be discussed explicitly; rather, principal technological constraints will be examined.

Previously, it was hypothesized that the reliability and efficiency of traffic networks can be enhanced by making the system dynamically responsive to individual user requirements. Also, favorable benefit-to-cost ratios were depicted for a dynamic system deployed on a high entropy network. Now, the major technological constraints on our present highway navigational system, which must of course be removed to provide a dynamic, user-responsive system, can be examined. As indicated earlier, these constraints are as follows:

- Open loop communication.
- Utilitarian, rather than individualized communication.
- Separation of roadway and vehicle components of highway transportation.

Communication is *open loop* in the sense that information flows in one direction only—from the highway sign to the vehicle operator. There is no provision for the driver to make his destination known to the system. In other words, there is no feedback channel in the system.

In any control system the consequence of open loop operation, with no feedback, is that very careful calibration of the system is required. In the present route guidance system this calibration is accounted for by the design of the sign message, which leads to another major constraint nonindividual or generalized, presentation of routing information.

Information conveyed by signs must be designed for the traffic stream at large. Consequently, it is inherently impossible with sign messages to convey precise meanings according to individual driver's needs or destinations.

If a sign message does not conform to a driver's expectations or if he lacks good orientation, the result is hesitation and indecision at crucial decision points. Consequently, accident potential is increased at decision points, and turbulence can be introduced in the traffic stream with resultant adverse effects on capacity. A wrong decision means extra travel and driver frustration. Also, there are documented cases of drivers backing down freeway exit ramps and per-

[illegible] hazardous weaving maneuvers after they had decided that the wrong decision had been made (*26*).

There is another constraint on our present highway system: vehicles operating in the system and the operators of the system—highway authorities—are relatively independent of one another. In today's highway system, this independence is manifested by the lack of direct communication links between the highway and the vehicle.

For example, Desrosiers (*25*) has shown that a considerable period elapses before vehicle operators adjust to a change in the speed of progression of a signal system. His research clearly shows the need for direct communication of the traffic signal setting to operators of vehicles using the system.

In summary, by removing the constraints on communications between vehicle and highway, many benefits, other than route guidance, could ensue. However, description here will concentrate on how a two-way communication system between the vehicle and highway can benefit the route guidance function.

Techniques and system solutions for vehicular routing

The number of techniques for communicating directional and guidance information to drivers has increased as technology has grown. Early constraints have become less compelling as the economics of electronic circuits have become less restrictive from the user's point of view. A brief look at highway-signing methods and routing-communication techniques permits a perspective that leads to solutions to highway routing problems that should be both economically feasible and socially acceptable.

Through the years, highway route signs and marking techniques have changed tremendously. From the era of makeshift local directional signing and colored bands on poles to identify routes, signing has progressed through the establishment of the U.S. and State route signing to the standardized Interstate system signing. Whether every change has been truly an advance is questionable. Color coding, currently being suggested in some quarters as a step that should be taken to promote smooth flow and safety, is characteristic of some of the earliest route markers. Following is a brief description of some of the most prominent methods of signing in use, or experimentally operated, today.

Static visual signing. Static visual signs are attempts to provide the driver with destination and routing orientation. Occasionally, with the assistance of maps, the driver may choose, one, two, or three routes to a specific destination. Destination information, if it appears on the sign, does not necessarily relate to the driver's destination, but may be merely another milepost to the driver's ultimate destination. The routes given to the driver do not always take into account delays caused by [illegible] transient events. They [illegible] travel time, for [illegible] On high speed road- [illegible] the driver, but their size creates a driving hazard. In urban areas signs may tend to be very small and difficult to read even at low speeds. Sign designers, however, do not take into account the variable visual proficiencies among drivers or the complicated maneuvers that often have to be performed. Thus signs are frequently not placed in an optimum location to aid all drivers.

Variable message visual signing. In recent years variable message signing has been used to a limited extent. This type of visual signing attempts to take into account delay owing to congestion. By proper sensing equipment or by use of a clock to indicate the beginning and end of peak periods, delays can be indicated by some figure of merit, and possible alternate routes transmitted to the driver by the variable message sign. Again, this type of signing is merely an intermediate aid to the driver and does not account for his ultimate destination.

Audio signs. Studies of audio signs have shown that advisory information regarding approaches to exits, obstacles, maintenance operations, traffic accidents, etc., can be transmitted to vehicles as they proceed down a highway. This system uses a prerecorded audio message that is continuously repeated at preselected points on the road. A roadside antenna, trigger loop, and transmitter are employed, as well as an in-vehicle receiver. This type of signing is conceptually similar to static and variable message signing; however, the system could be portable and act as an early warning device for drivers as they approach hazardous situations. It is conceivable that alternate-route information could also be prerecorded.

Direct inquiry techniques and maps.—A driver today has several methods at his disposal for obtaining a routing to a destination. The familiar oil company maps as planning devices and tourist services offered by oil companies and automobile clubs are the most common assistances given to drivers. In some areas he may also call a travel service if he is equipped with Citizens Band Radio. All of these methods still require the driver to search out the appropriate signs of landmarks.

Passive communication systems.—In a passive communications system, the roadside equipment would continuously transmit coded signals concerning all destinations. The vehicle equipment would accept only the signal that corresponds to the encoded destination. The coded signal would also correspond to the type of maneuver to be performed at that particular intersection or trigger a particular message to be presented, either visual or aural, to advise the driver of the maneuver to be performed. This system would require that most of the equipment be housed in the vehicle. If a number of nodes instrumented are very near to each other, the problem of radio interference arises. Many frequency allocations would be necessary, and these are almost impossible to obtain because the spectrum at present is fairly well filled. Another disadvantage is that maintenance of complex equipment would be the responsibility of the driver or vehicle owner, if the logic equipment is housed in the vehicle. From the condition of some of the automobiles on the road today, it is difficult to visualize communication equipment that is always in working order. Gumacos and Cramer have reported on such a system (*27*).

Active communication systems. In an active communications system, the vehicle transmits a coded destination to roadside decoding equipment through a two-way, near-field communication link. On request from a decoder, the in-vehicle transmitter is triggered to send the destination to the roadside equipment which *looks up* the destination in a preprogramed, *best route* solution matrix or set of tables and sends, through the same communication link, the appropriate coded signal to correspond with the proper maneuver to be performed at that particular intersection. The maneuver symbols, or messages, are activated in the vehicle by the coded signal, which triggers the appropriate display elements. The bulk of the equipment is at the roadside; the in-vehicle equipment is simpler and requires less maintenance. With a vehicle active system, benefits are numerous. For example, origin and destination study capability, traffic count data, traffic surveillance capabilities, etc. are built in. Also, a near field communication link requires a minimum of frequency allocations from the Federal Communications Commission. The best example of such a system is described in the final report of a recent Public Roads research project (*28*).

A Proposed Route Guidance Concept

About two years ago, the Bureau of Public Roads began to develop a new route guidance concept. A number of studies that indicated positive benefits from a system to overcome some of the deficiencies of existing route guidance techniques, had been concluded or were nearing completion (*12, 22, 27*). Furthermore, there were proposals from a variety of groups to proceed with development of devices for directing information to drivers by various means. Though none of the devices proposed were actually complete route guidance systems, many of them had features that possibly could be employed in a complete system. It was becoming evident that a comprehensive analysis and plan was needed to integrate previous research and to guide future work. From the integration of previous work and a thorough analysis of the problem, a proposed route guidance concept has evolved. Concept testing of a closed loop, individualized, integrated-highway-vehicle communication system was carried out in the Washington, D.C. area. The results indicated highly significant improvements in travel time and stress reductions (*29*). The essential features outlined in the succeeding discussion characterized the system concept.

Individual communication

To overcome the inherent limitations of highway signs and their messages to the traffic stream at large, it was deemed necessary that the route guidance system communicate information to individual drivers. This feature is justified by studies[5] (*6*) that have shown that drivers' control performance is facilitated when uncertainty is decreased and that have suggested that traffic operations should be significantly improved when individualized communication is employed at freeway exit ramps.

Specific maneuver information

To bring drivers' decision-making requirements to an irreducible minimum, it was apparent that information on the specific maneuver to negotiate a choice point must be conveyed, and that it be conveyed unambiguously to individual drivers. Therefore, the concept requires that the communication must terminate in the drivers' vehicles, which in turn, creates a need for an information display for the driver. The driver's display is the final link in the communication subsystem, and the information it emits is the basis for the driver's control actions. The design of the display is critical, as it is the major informational interface between the human operator and the highway-vehicular environment. A detailed analysis of requirements for relating displayed information to highway geometrics has been given by Eberhard et al. (*30*). Human factors analysis has indicated that the best display technique available is the *head-up* display being used in low altitude aircraft. A feasibility study was conducted and eventuated in a vehicular head-up display prototype (*31*).

A parallel requirement to that of providing specific maneuver information at a choice point is: the sequence of choice point decisions must add up to a *best route* to a driver's destination.

Unique codification schemes

The route guidance concept that has evolved requires information coding schemes that are desirably efficient, flexible enough for other highway uses, capable of future expansion without disruption, and compatible with state-of-the-art of information-handling subsystems, both machine and human. A coding scheme for intersections, which is compatible with *best route* solutions and which is human engineered to reduce short-term memory requirements, is described in the final report of a Public Roads study (*32*).

Efficient machine code techniques and optimum human usage is required in the interface between address machine logic and the operator during the process of encoding. Some of these requirements have been addressed by researchers, and analysis is being continued by the Bureau of Public Roads.

According to analysis of the character of directional codes, links, nodes, and link scalar quantities are insufficient for high quality solutions of *best routes*. Solutions of large networks having deterministic link scalars can be obtained in reasonable periods using decomposition techniques, but most routing problems have relatively small stochastic link scalars as input data, which suggests more microscopic analysis would be fruitful.

Models of traffic flow using a combination of parallel links and sequential and conditional dependencies to select particular links at each node have not been formulated. High quality solutions will depend on a codification system yet to be devised. Further, as a result of coding work accomplished thus far, it is apparent that information coding goes beyond the route guidance concept and overlaps with several other important highway functions, including urban traffic control and urban transportation planning. The present coding scheme will be the subject of continuing review, and compatibility analyses will be made with other highway functions in mind.

Static to dynamic conversion capabilities

In the early stages of route-guidance-system development, it was recognized that a static system would have limited utility because a large part of highway travel consists of repeat trips over familiar routes. For the guidance system to be useful for these trips, it would be necessary to sense changing traffic and environmental conditions, and accordingly, change routings to provide *best routes* in a dynamic situation. Analysis presented earlier in this article provides the rationale for such a decision, and one of the design goals has been to develop coding and hardware that is capable of conversion to a dynamic system. Design of the system should provide for simple methods to update preprogramed stored tables at each instrumented roadway area.

Compatibility with existing signing

Another feature of the proposed route guidance system is its inherent compatibility with existing guidance techniques. Present signs would remain in place during conversion to the new concept and would not have to be changed in any way. After the new concept was widely implemented, they could be reduced to the minimum that might be required for back up if the system fails or for the fraction of vehicles or intersections that might not be equipped.

Warning and regulatory functions of present signs can also be handled by the proposed system, including some situations that are troublesome for existing signs. One example is the familiar *lane drop* situation at interchanges. The proposed system can give a driver an advisory lane-change-maneuver signal at any desired point along the highway. Properly placed for traffic and geometric design conditions, this signal could eliminate driver indecision and provide ample time for a merging maneuver. Moreover, the left hand exit ramp could be handled similarly. The one-way street situation is an example of a regulatory-sign function that could be served by the proposed system. One-way streets would be accounted for and drivers would get only signals for the proper direction. The problem is nontrivial for streets that are operated reversibly during peak hours; the roadside logic could respond to the change in direction and relieve the driver of reading and interpreting reversible one-way signs.

Programatic Development

No matter how superior the concept is, without a program of research and development studies with reasonable levels of support, the effort of identifying a desirable roadway navigational system is merely a mental exercise. Development of a program based on the findings of other investigators, as well as inventive attempts at communicating with vehicles from the roadside, has begun. What to communicate, when to communicate, how to communicate, and why we should communicate has been presented herein in considerable detail.

Guidelines were developed in an intensive study of the highway coding and route recognition problem conducted at the Bureau of Public Roads in 1966.[4] Within 6 months, a research and development plan that incorporated the following phases was developed:

- A detailed analysis of driver information needs and rules for the optimal transfer of such information.
- A coding requirement and format.
- Development of a programed routing system design for both roadway and in-vehicle hardware.
- Construction and installation of a limited amount of hardware for test and evaluation.
- Conduct of a test and evaluation plan.

The plan was to serve as the nucleus of major study, but could not be considered a program in itself. Time phasing and identification of the criticalness of each step ensued. Since that time 26 research and development functions have been identified.

Driver information requirements for route guidance

Development of a rational description of highway routing was the first logical step in providing a highway guidance scheme. There was no evidence that such a description had been developed for signing applications, despite the substantial costs of signs estimated to be in excess of $3 billion.[4]

A generic language for highway routing applications was developed as part of an effort to define the navigational part of the driving task (*31*). Basically this was an information-requirements study, but it was developed at a fair level of detail.

A practical consideration has evolved from the question of where, spatially, should information be presented to drivers. An analysis of information lead-distance followed from work formerly conducted (*22*). Rules for decisions on where roadway hardware should be physically located with respect to highway choice points are being formulated. Most of this work is based on empirical analysis of the need for specific maneuver information, an analysis that is to be verified by use of vehicular stability measures.

Many factors influence the variability of responses elicited by different drivers, hence the influence of aging, stresses, and specific information requirements of traffic and highway geometrics all are being studied.

The task of establishing driver information requirements includes some prediction of human acceptance of a system in which design parameters, hopefully, are optimized. Preference data are being collected from a substantial population representing a cross section of drivers. A unique form of questionnaire, basically a motion picture technique, is being used to obtain these data. All of this work will lead to development of a final routing configuration that is engineered for human use.

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[5] Additional Studies on Driver Information Processing, by J. W. Senders and J. L. Ward, Final Report, Contract CPR-11-5096, Bureau of Public Roads, 1968. (Unpublished).

Attributes of network coding and best-route algorithms

As early as 1964, the Public Roads staff recognized the need to develop a national standard system of coding highway nodes that would be equally available to all potential designers of electronic routing and highway communication systems. A computerized technique was developed to solve the problem of routing an individual from any origin to any desired destination within a highway network (27). The uniqueness of the code and also the partitioning scheme for dividing up the highway network are shown by the following explanation:

Uniqueness.—A coding system and formatting technique has been developed for the unique identification of more than four million intersections in the United States. It is a logically consistent method for naming intersections and roads of a highway network.

Partitioned sets.—Owing to the size of the existing and projected roadway network, a certain procedure was used to develop the code. As the code had to properly identify the roadway network, a dual form of partitioning was established—geographic, to describe locality by the code; and hierarchial, to reduce the total information loading and handling requirements by selective transmission of hierarchial information. This is done by furnishing complete information about the network in the driver's immediate vicinity and less information about the network, at locations removed from his vicinity.

Rapid access and updating. An algorithm (27) enables a solution of a matrix to determine *optimal routes* on a highway network. Optimal routes can be described by such criteria as travel time or distance. Use of such a code in a programed routing system requires three matrix solutions—one for each of the two peak and the off-peak periods. However, in a dynamic system, solutions are needed more frequently to account for delay, congestion, weather, etc. Therefore, a system for rapid updating and access must be developed for the dynamic system. Algorithms are yet to be developed to determine optimum real-time solutions for alternate routes in a network.

Hardware design

Based on the route guidance concept described and the *best* route algorithm and network coding, hardware to implement the system was designed. An engineering model, now operational was developed. The final report of the development project (28) gives details of the system design. Only the general features will be described here.

The [illegible] include both vehicular and [illegible] components. The vehicular component [illegible] destination. [illegible] mit the destination to the roadside components at an intersection, and display the correct maneuver symbol or message received from the roadside.

The roadside component receives the driver's destination code, decodes the destination in terms of a specific maneuver symbol or message, and transmits the maneuver information back to the vehicle. Roadside decode logic is based on stored programs developed from solutions to the *best route* algorithms mentioned earlier.

Communication occurs through simple loop antennae mounted under the vehicle and buried in the pavement in each intersection approach lane. During the period (0.03 sec. or less) when the vehicular antenna is within the field of the buried road loop (generally corresponding to the physical boundaries of the road loop), destinations are transmitted and maneuver instructions received. Thus, each vehicle is specifically and individually serviced by the system.

Testing and evaluation of system effectiveness

The route guidance system described here is being proposed as a means to improve the safety and efficiency of the entire highway transportation system. The implementation of such a system would require large expenditures by highway authorities and road users, and before a decision to implement such a system is made there must be a sound and convincing demonstration that the expenditures can be justified; in other words, do the benefits justify the costs?

To provide inputs for a benefit-cost analysis, an elaborate test and evaluation program is being planned. The goal of the program is to evaluate the effectiveness of the system from the standpoints of both the driver and the highway authorities. Accordingly, the system evaluation plan consists of two distinct types of tests which can be classified as macroscopic and microscopic, as follows:

Macroscopic tests

Driver benefits:

- Time savings owing to *best routes* and fewer errors.
- Fuel savings.
- Fewer accidents.
- Fewer information stops.

Road benefits:

- Minimize overall travel time.
- Efficient use of network.
- Reduced congestion.
- Reduced air polution.
- Less congestion from accidents.
- Fewer roadside hazards.
- Fewer accidents.
- Improved aesthetics.
- Reduced signing requirements.

Microscopic tests

Driver benefits:

- Reduced operator stress.
- Improved vehicular control owing to uncertainty.

Road benefits:

- More efficient use of highway network because of decreased turbulence at decision points and higher overall speeds.

Plans for conducting these tests call for a network of about 100 intersections in a section of a large metropolitan area and a fleet of about 50 instrumented test vehicles. Some tests will be conducted entirely on the instrumented network; others may consist of test runs in other noninstrumented portions of the area. Network and driver characteristics and driver population will be selected to make the tests as representative as possible. Travel characteristics, such as trip length, trip purpose, and time of day, will also be considered in the formulation of the experiments.

The test and evaluation program is being designed to produce definitive data on benefits that can be expected from a static route guidance system. The test network is being selected to provide several configurations in which dynamic system concepts can be evaluated. The costs of equipping and programing the network and the test vehicles will also be established.

It is hoped that the results of the test and evaluation program will serve as inputs to a comprehensive benefit-cost analysis. The ultimate objective of the test and evaluation program is to determine the feasibility of wide-scale implementation of the system, which would, of course, take into account developments of related systems for aiding the driver.

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(9) *Interstate System Accident Research, Study II, Interim Report II*, by Julie Anna Cirillo, PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, vol. 35, No. 3, August 1968, pp. 71–75.

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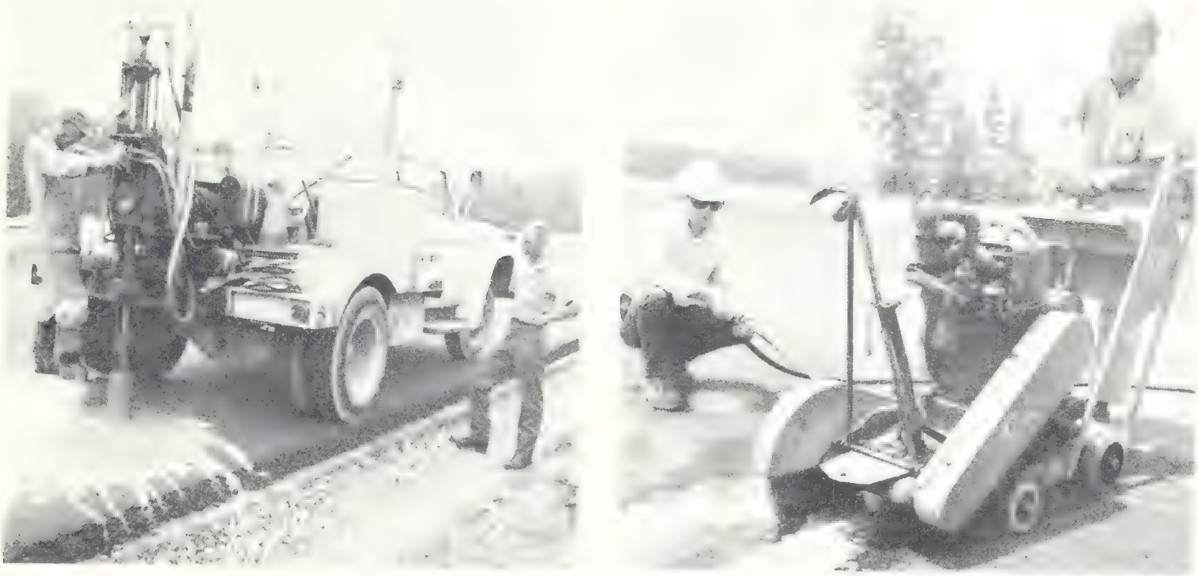

Samples being taken from bituminous pavement for testing—coring a 6-inch diameter sample (left) and sawing a square sample (right).

Quality Assurance in Highway Construction

Part 4—

Variations of Bituminous Construction

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BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

This is the fourth part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, Part 2.—Quality Assurance of Embankments and Base Courses, and Part 3.—Quality Assurance of Portland Cement Concrete were presented in previous issues of PUBLIC ROADS. The remaining parts, to be presented in succeeding issues, are 5.—Summary of Research for Quality Assurance of Aggregate, and 6.—Control Charts.

Introduction

IN 1879, a natural asphalt from Trinidad was used on a street project in Washington, D.C., marking the first modern use in this country of a bituminous material in road construction. The practice of treating the uppermost surface of roads with a thin bituminous overlay, such as that used on this street project, continued. The advent of the automobile, bringing with it the production of gasoline and its byproduct, bitumen, brought the material into widespread use, first as a dust preventative and later as a binder for asphaltic concrete.

In the early stages of bituminous mixture development, many of our present specifications and tests were developed to guide contractors and to provide rules for acceptance. Initially, one of the major functions of a specification was to supply technological instructions to the contractor and field engineer. It was necessary to specify exactly how to produce the mixture, how to place it, and how to compact it. Now, the industry has progressed so well that the States soon should be able to specify characteristics of the final product in terms of measurable parameters and to accept it when test results indicate that desired characteristics have been obtained. Before this goal can be reached, however, some problems must be overcome and the ultimate degree to which *end result* specifications can be used in bituminous construction must be determined. Nevertheless, progress is being made in changing from the contractor-State-control construction team to the true contractor-control and State acceptance concept. In shifting responsibilities it is important that acceptance plans protect both the contractor and the State.

To determine the quality characteristics of current construction, many States have been measuring variations in accepted bituminous production. Most of them have followed the guidelines developed by the Bureau of Public Roads Quality Assurance Task Force. The studies patterned after these guidelines not

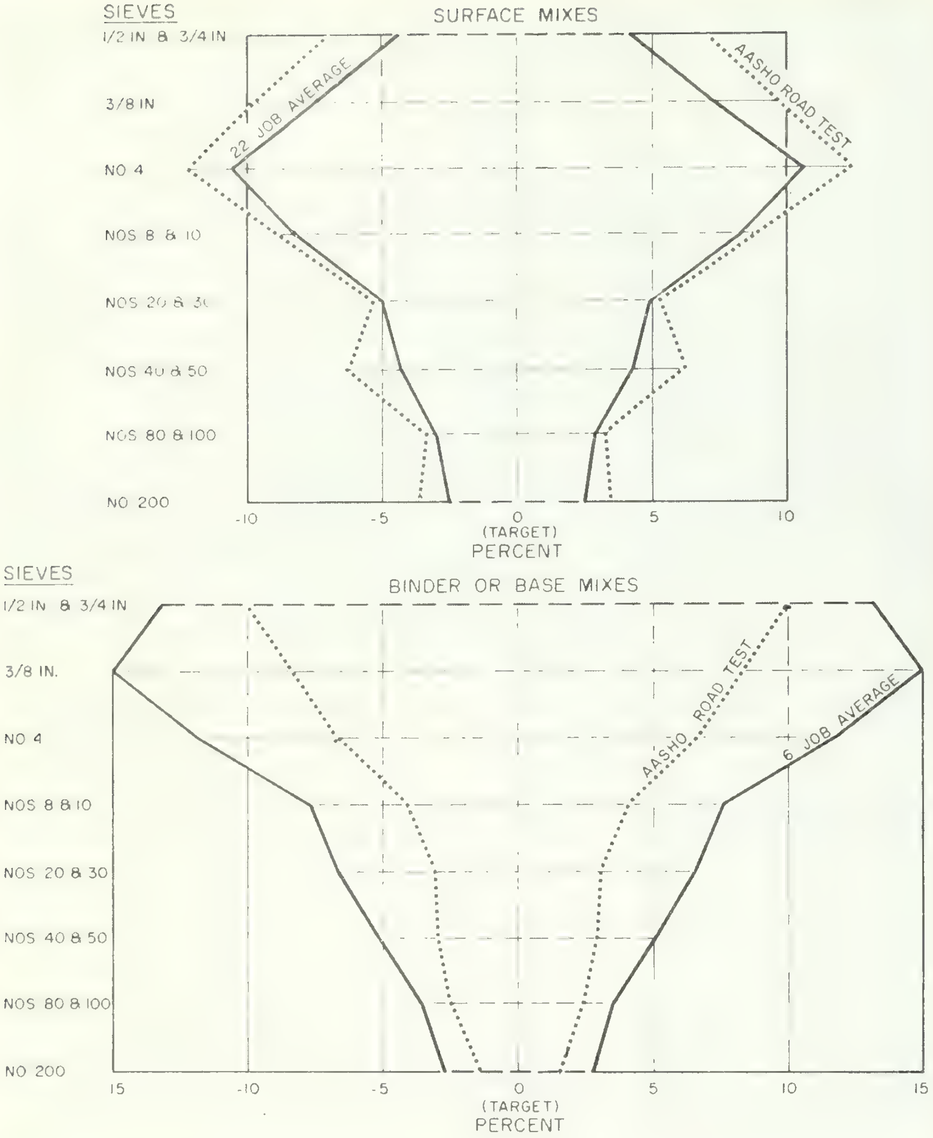

Figure 1.—Computed 3$^\sigma$ limits on target values of job mix formula for aggregate gradation—surface and binder or base courses.

only are providing estimates of the quality of construction, but also are isolating causes of variation.

Data from these studies are revealing the following significant results:

- Variability, indicated by the standard deviation, σ, is itself a variable, and a set value for a standard deviation applicable to the process cannot always be assigned.
- Calculations of the amount of material, or construction component, within present tolerance limits often indicate a considerably lower percentage within the tolerance limits than is expected.
- Test variation, or test error, is often an important factor affecting acceptance or rejection of the material.

Many laboratory-designed tests and sampling plans now being used for on-the-job control and acceptance are inadequate. These devices, developed for 1940 production rates, are still being used to attempt to control and accept bituminous production that exceeds [illegible]

Research results obtained in studies of construction variations by several State highway departments have been summarized in this part. Compiling data from bituminous hot-mix projects throughout the country is like putting together a jigsaw puzzle in which the pieces never quite fit and even some are missing. Certain editorial privileges and mathematical manipulations were used to present data uniformly. Sometimes statistical rules were not strictly adhered to. For example, standard deviations, σ, were presented as an arithmetic *average* of individual project results. Components of variance were similarly handled. The term *averages* for data of this type was used to avoid ambiguity with other averaged data. Properly, variance data, in which the square root is directly or indirectly involved, should be *pooled.*

Pooling consists of summing the squared standard deviations (variance) multiplied by the number of test results per project, n, less one $\sum[(\sigma^2)\ (n-1)]$, dividing by the total number of test results from all the projects, n, less the number of projects, N, $(\sum n-\sum N)$, and extracting the square root. The *pooled* standard deviation for the No. 4 sieve in table 1 was 3.56 percent, compared with 3.51 percent obtained from an arithmetical average. From an engineer's standpoint the difference, $3.56-3.51\ (=0.05)$ is considered insignificant. Similar comparisons for other *average* standard deviations, σ, showed a similar difference. This insignificant difference is to be expected, as each project value was obtained from approximately 200 test values and a standard test procedure.

Aggregates

Aggregate represents the largest percentage of any ingredient in a bituminous mixture; consequently, aggregate characteristics significantly control the characteristics of the pavement mixture.

Laboratory research and field experience indicate that, although gradation within the

Table 1.—Averages of aggregate gradation data from extraction tests

Sieve size	Average standard deviation ($\bar{\sigma}$) of percent passing	Shift of average (X) from job mix target	Average variance components as a percent of total variance (σ_0^2)			Computed average compliance with job mix tolerances
			Testing	Sampling	Material	
Surface mixes, 22 projects						
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
3/4 in. or 1/2 in.	1.43	1.70	72	4	24	99
3/8 in.	2.49	1.73	29	31	40	93
No. 4	3.51	2.95	12	18	70	78
No. 8 or 10	2.81	2.45	10	15	75	77
No. 20 or 30	1.74	2.10	13	18	69	87
No. 40 or 50	1.37	1.72	18	15	67	87
No. 80 or 100	1.00	1.44	17	11	72	82
No. 200	0.94	1.43	21	14	65	74
Average .... pct.	1.91	1.94	24	16	60	85
Base or binder mixes, 6 projects						
3/4 in. or 1/2 in.	4.33	1.66	65	13	22	83
3/8 in.	4.93	5.88	55	30	15	60
No. 4	3.92	2.03	46	17	37	76
No. 8 or 10	2.53	1.81	19	13	68	50
No. 20 or 30	2.17	2.22	25	28	47	81
No. 40 or 50	1.67	1.63	23	31	46	84
No. 80 or 100	1.15	1.23	30	30	40	97
No. 200	0.88	1.02	21	14	65	74
Average .... pct.	2.70	2.19	36	21	43	76

confines of a rather wide grading band is necessary to produce a high-quality product, no single gradation can be adopted as the ideal one for bituminous mixtures. The gradation to be specified depends on the type of surface desired. The maximum size stone used in the pavement is also influenced by the availability of aggregates. The best combination of various sizes then becomes a design problem leading to the establishment of a job-mix formula. The job-mix formula also includes the desired asphalt content.

Under present practice, the State often accepts responsibility for determining the job-mix formula. Once the job-mix formula is established and approved by the engineer, it becomes the target or central value for process control. A tolerance is usually included to account for normal variability of materials or processes.

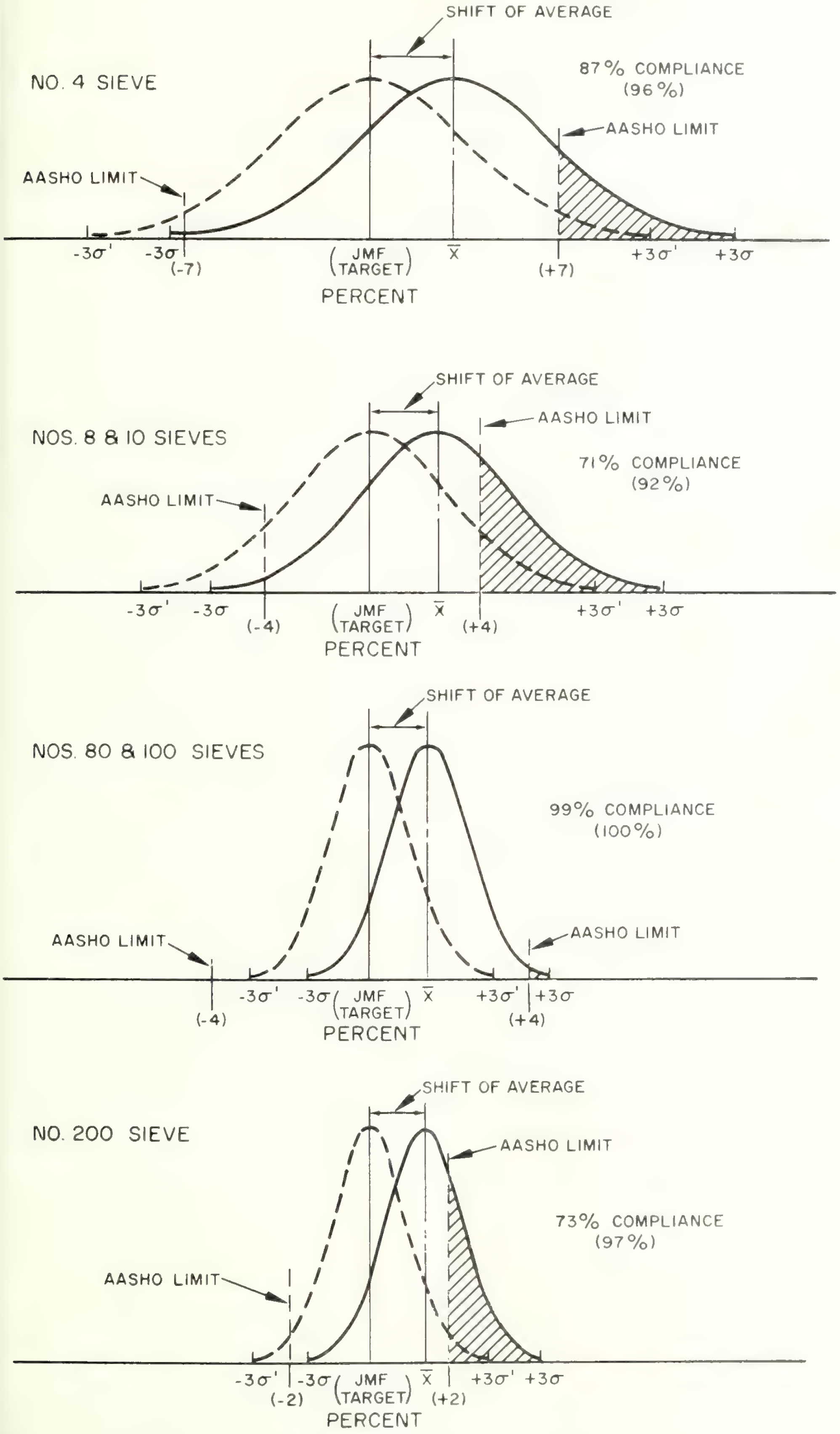

Figure 2.—Theoretical-frequency-distribution curves of gradation data for selected sieves.

Variations in aggregate gradation

According to the research studies being conducted, randomly selected samples, taken independently of control samples, usually show deviations in gradations that often are larger than the specification tolerances. Summaries for each aggregate gradation in presently accepted construction and their relations to specified tolerances are shown in tables 1 and 2 and in figures 1, 2, and 3. Data for surface course mixes are included from 22 projects in eight States and for binder or base mixes from six projects in five States.

A consolidation of gradation data for aggregates from extraction test results is shown in table 1. These data were obtained on samples taken independently of those used for job control and acceptance. Departure of averages, $\overline{X}$, from the job-mix formula for individual jobs were about evenly divided below and above this target.

An analysis was conducted to determine the components of variance that could be attributed to sampling, σ_s^2, testing, σ_t^2, and materials, σ_a^2. For surface material, combined testing and sampling variances $(\sigma_t^2+\sigma_s^2)$ are shown to be in the range of 25–35 percent of the total variance, (σ_o^2), for sieve No. 4 and smaller sieves. For the larger sizes of either surface or base course materials, the combined sampling and testing error was a significantly larger proportion of the total variance. For the base course materials, even smaller sizes showed large sampling and testing variances. The statistically computed average percent compliance to States' job-mix formula and tolerances are also shown.

Further analysis of the data from the construction projects for surface course materials is given in table 2. These data provide a summary of variations from the least variable one-third, and the most variable one-third of the projects, as well as the average for the total. Also, statistically computed percent compliances with suggested tolerance limits of the AASHO Guide are shown instead of computed conformance to job-mix tolerances. In general the most variable projects show average standard deviations, $\bar{\sigma}$, of about twice the corresponding values for the least variable projects.

The plus or minus three average standard deviations, $\pm 3\ \bar{\sigma}$, (table 1) for both surface and binder or base courses are plotted in figure 1. The bulged shape, or the largest spread, emerges at the No. 4 sieve for surface courses, and at the ⅜-inch and larger sieves for binder and base courses. Average variations for base and binder courses are about ⅓ larger than those for surface courses. Superimposed on each diagram are $\pm 3\ \sigma$ values for each

Table 2.—Averages of surface course aggregate data from extraction tests on 22 projects

Sieve size	Average standard deviation percent passing			Deviation of average ($\overline{X}$) from job-mix target			Suggested AASHO tolerance limits	Computed compliance with suggested AASHO tolerance limits		
	1/3 of jobs having least variable σ	All jobs	1/3 of jobs having most variable σ	1/3 of jobs having least variable σ	All jobs	1/3 of jobs having most variable σ		1/3 of jobs having least variable σ	All jobs	1/3 of jobs having most variable σ
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1/2 in. or 3/4 in.	0.81	1.43	2.32	0.53	1.70	2.62	±7	100	100	95
3/8 in.	1.71	2.49	3.46	1.06	1.73	2.36	±7	100	98	91
No. 4	2.33	3.51	4.52	2.22	2.95	1.83	±7	98	87	68
No. 8 or 10	1.90	2.81	3.85	1.68	2.45	4.04	±4	89	71	49
No. 20 or 30	1.32	1.74	2.24	1.70	2.10	3.06	±4	96	86	66
No. 40 or 50	0.93	1.37	1.82	1.41	1.72	1.23	±4	100	95	94
No. 80 or 100	0.65	1.00	1.36	0.65	1.44	1.48	±4	100	99	97
No. 200	0.51	0.94	1.45	1.43	1.43	1.74	±2	87	73	57
Average ......pct.	1.29	1.93	2.64	1.33	1.94	2.67	....	96	89	77

type of mix from the AASHO Road Test (*1*).[1] Because the construction of the AASHO Test Road was very carefully controlled, these data are considered a solid base with which to compare research data. For average surface course data, almost perfect agreement is shown with the AASHO Road Test results. There is no apparent reason why AASHO Road Test gradation data show less variation (smaller standard deviation) for base courses than for surface courses.

Both the standard deviation and the shift of the average, $\overline{X}$, from the job-mix target affect conformance to specifications. The effect of $\overline{X}$ shift from the target value is shown in figure 2, in which values from table 1 were used to compute theoretical normal frequency distributions of four selected sieve size groups. Darkened areas of the tips of the curves represent noncompliance with suggested (*2*) AASHO Guide Specifications. The dashed line curves, which are the same distributions shown by the solid curves, are superimposed on the job-mix target values. The percent compliance with AASHO limits of the superimposed curves are usually much larger than that of the solid curves, as is indicated by the figures in brackets.

Many consider that a reasonable conformance to a specification is met if tests indicate that 95 percent of the material is within the stated tolerance. For a normal distribution this 95-percent conformance level approximates the two standard deviation limits. For the 22 surface course projects, the relation of the spread of gradation represented by ±2σ from the average to the suggested limits of the AASHO Guide Specification (*2*) is shown in figure 3. The ±2σ range is also shown for the 1/3 of the jobs with the largest standard deviation. Except for No. 8 and No. 10 sieve group, the ±2σ range of all jobs are within the AASHO Guide Limit. The ±2σ ranges for the third with the largest standard deviations are relatively close to the AASHO limits; the most significant deviation occurred again for the No. 8 and No. 10 group. There were also significant deviations for the No. 4 and the No. 200 sieves.

The relation of the standard deviation to the [illegible] percent retained on each sieve is shown in table 3 for the different groups of surface and base or binder mixes. It will be noted that the standard deviation for surface course mixes seems to be related to the amount of material retained on each sieve, as shown in figure 4.

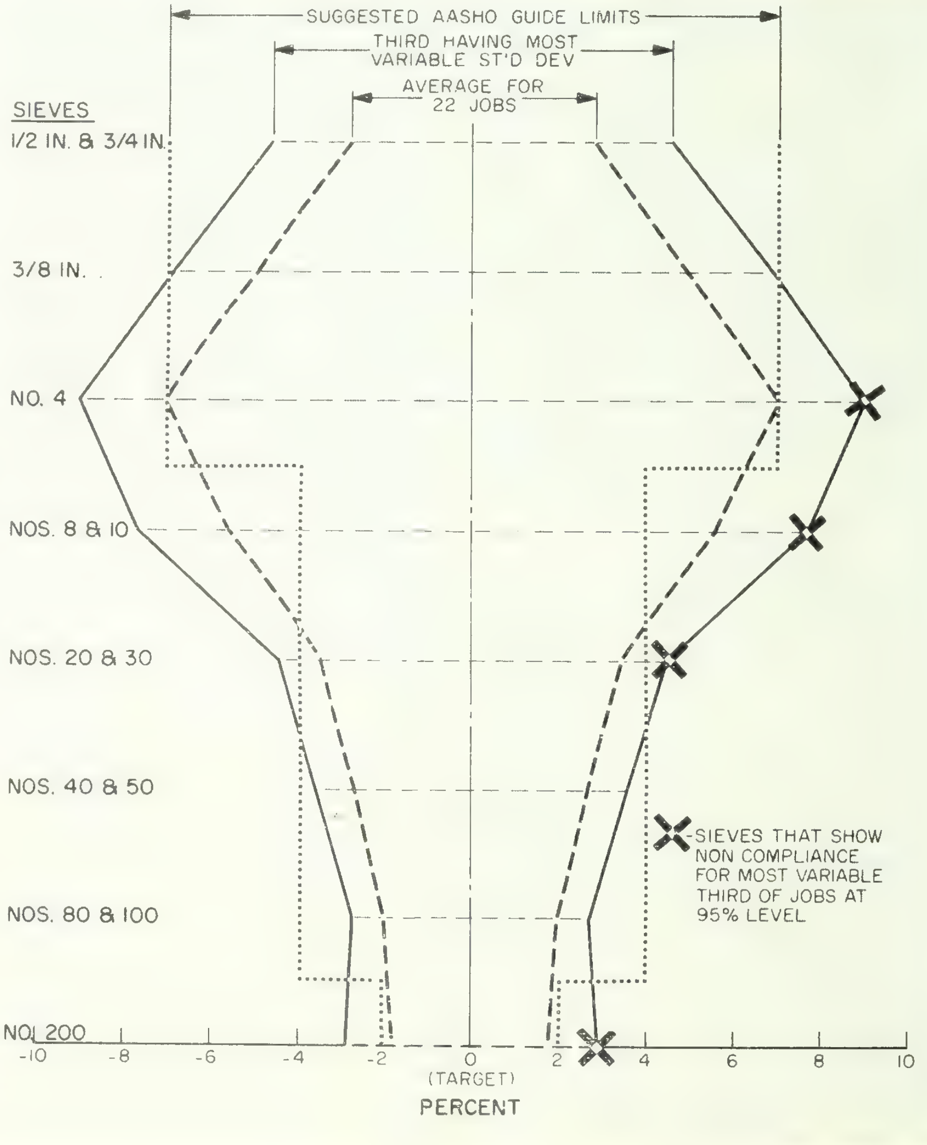

Figure 3.—Comparison of surface-course-gradation 2σ limits and suggested AASHO guide limits.

Asphalt Content

The quality and quantity of asphalt in a pavement mixture largely determines the useful life of the pavement, provided that the pavement has been properly compacted. Too much asphalt in a mixture can cause flushing and rutting of the pavement, and too little asphalt can cause cracking or raveling. Thus, a close control of asphalt content is desirable.

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[1] Italic numbers in parentheses identify the references listed on page 211.

Asphalt content data from extraction tests on 26 surface course mix and seven base course or binder mix projects are arranged in table 4 according to size of standard deviation. Also, the surface course mix projects are grouped by thirds to delineate those with the least variable standard deviation, the middle third, and those with the most variable standard deviation. Shown in separate columns are the plus or minus shift of the job average from job-mix target and statistically computed compliance with ±0.4, ±0.6, and ±0.8 percent tolerances, respectively.

The average $\bar{\sigma}$ of extracted asphalt for surface mix projects was 0.28 percent. The average for binder or base mix projects was 0.35 percent. The computed $\pm 3\sigma$ limits for 20 of the 26 surface course projects, in which the job-mix target value was reported, are shown in descending order in figure 5. Also shown in figure 5 is the shift of the average from the target value. The asphalt content for Project No. 1 was on the target; it was also the only project to show variations that were less than the suggested ±0.4 percent limits of the AASHO Guide. The three standard deviation limits for individual projects ranged from 0.36 to 1.59 percent. The computed 3 σ limits for AASHO (*1*) and WASHO (*3*) road tests were 0.54 percent and 1.20 percent, respectively. On about ⅔ of the jobs, the job averages were lower than the target (table 4). Only three surface mix jobs complied 100 percent with assumed tolerances from the job-mix formula of ±0.6 percent, although half the total showed more than 95 percent compliance. Increasing the tolerance to ±0.8 percent did not appreciably increase the number of jobs having more than 95 percent conformance.

Data for the surface course projects from table 4 are shown in figure 6, grouped into three sections according to standard deviation of asphalt content. The $\bar{\sigma}$ for each group was used to construct the three normal curves, which show that the most variable projects also had the largest shift of the $\bar{X}$ from the target value. This shift indicates a lack of job control that adversely affects both the average and the variance. The computed conformance percentages are based on the AASHO's suggested ±0.4 percent tolerance. In parentheses, beneath the percent conformance for each group, is the computed percentage that would have been obtained if all projects averages had been on the target value.

Testing and sampling variance

Testing, sampling and material variances of asphalt content for both surface and binder or base-course mixes are shown in figures 7 and 8, respectively. These variations, imply that results of a single extraction test are not a reliable measure of asphalt content. However, the precision of the measurement can be improved by using the average of several

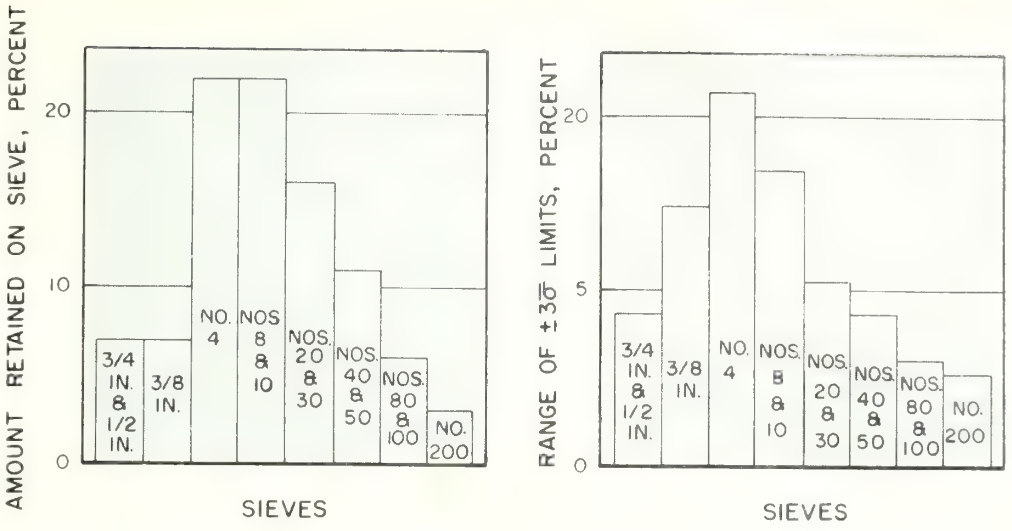

Figure 4.—Comparison of average amount retained on sieves and range of $3\bar{\sigma}$ limits—surface course projects.

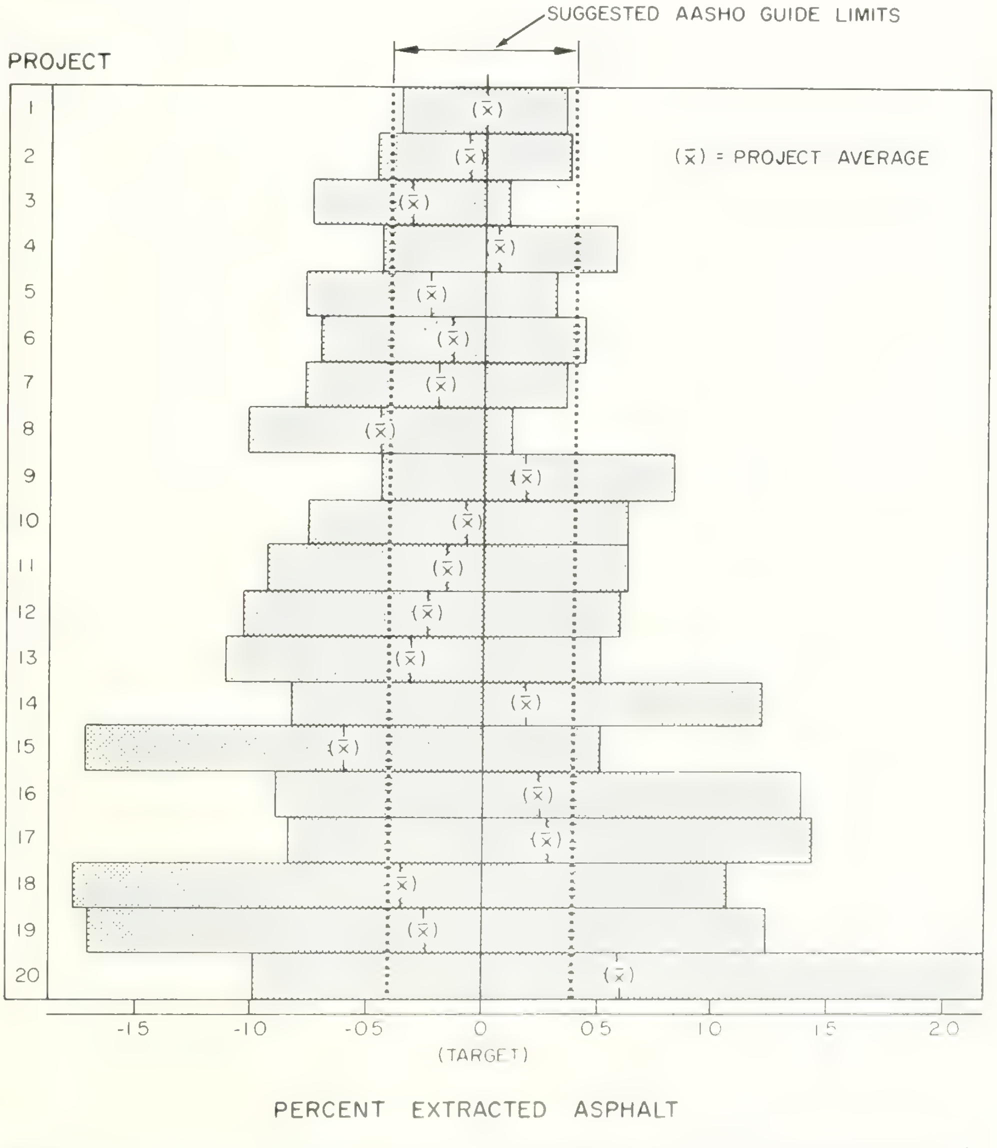

Figure 5.—Computed 3σ limits and shift of average from job-mix-formula target for extraction test data of asphalt content—20 surface course projects.

Table 3.—Average sieve data from extraction tests

Sieve size	Average amount passing [1]	Average amount retained on indicated sieve size [1]	Range of ±3 average standard deviation limits				Suggested AASHO guide (±)limits
			Surface course mixes			Base or binder mixes all jobs	
			Least variable third	All jobs	Most variable third		
	Pct.	Pct	Pct.	Pct.	Pct.	Pct.	Pct.
$\frac{3}{4}$ in. or $\frac{1}{2}$ in	93	7	[2] 2.4	[2] 4.3	[2] 7.0	13.0	7
$\frac{3}{8}$ in	86	7	[2] 5.1	7.5	10.4	14.8	7
No. 4	64	22	[2] 7.0	10.5	13.5	11.8	7
No. 8 or 10	42	22	5.7	8.4	11.5	7.6	4
No. 20 or 30	26	16	[2] 4.0	5.2	6.7	6.5	4
No. 40 or 50	15	11	[2] 2.8	4.1	5.5	5.0	4
No. 80 or 100	9	6	[2] 1.9	[2] 3.0	4.1	[2] 3.5	4
No. 200	6	3	[2] 1.5	2.8	4.4	2.6	2

[1] For surface mixture only.

[2] Within AASHO guide recommended tolerance limits.

Table 4.—Bituminous content data from extraction tests

Job No.	Standard deviation (σ)	Shift of average from job mix target		Computed compliance with tolerances from job mix target		
		Below	Above	Suggested AASHO guide tolerance ±0.4%	Assumed tolerances	
					±0.6%	±0.8%
Surface course mixes						
	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
Least variable third of jobs:						
1	0.12	0.00	0.00	100	100	100
2	0.14	0.31		74	98	100
3	0.14	0.04		99	100	100
4	0.17		0.07	97	100	100
5	0.18	0.22		88	98	100
6	0.19	0.44		42	80	97
7	0.19	0.13		92	99	100
8	0.19	0.20		85	98	100
9	0.21		0.20	83	97	100
Average, least variable third	0.17	0.18		84	97	100
Middle third of jobs:						
10	0.22					
11	0.22					
12	0.23	0.06		93	99	100
13	0.26					
14	0.26	0.15		83	96	100
15	0.27	0.22		74	92	98
16	0.27	0.30		64	87	97
17	0.27					
Average, middle third	0.25	0.18		79	94	99
Most variable third of jobs:						
18	0.33					
19	0.34		0.20	68	88	96
20	0.37	0.60		29	50	79
21	0.38		0.30	51	78	91
22	0.38		0.25	58	82	93
23	0.47					
24	0.47	0.35		50	68	83
25	0.49	0.23		54	73	86
26	0.53		0.20	52	71	84
Average, most variable third	0.42	0.30		52	73	87
Average, surface course mixes	0.28	0.22		72	88	95
Base or binder course mixes						
1	0.22					
2	0.27		0.33	60	88	96
3	0.28					
4	0.38					
5	0.38	0.24		62	81	93
6	0.43		0.13	63	82	93
	0.50					
Average, binder or base mixes	0.35	0.23		62	84	94

results as the test value. Better precision can also be obtained by improved sampling and testing procedures.

Testing Variations

Effect of sampling point

Engineers disagree as to whether the location at which a sample is taken affects test results. According to present practice, extraction test samples usually are obtained from the truck at the plant so that results can quickly be made available. Research has been performed to evaluate the effect of the sampling location. Average test results of samples from the truck and those of core samples from the pavement are listed in table 5. These data from 10 jobs indicated no significant differences between core samples and truck samples. The bar graphs in figure 9 substantiate that the point of sampling does not significantly affect the variances for asphalt content.

Ash correction

The extraction test for determining asphalt content includes an ash correction for insoluble material that passes through the filter. Because field laboratories do not always operate under optimum conditions, it is thought by some that the State should dispense with running the ash correction in field laboratories and substitute constant corrections determined by a central laboratory.

Several studies were conducted to determine ash correction variations in the field. In a Florida report (*4*), field laboratories tests, when compared with central laboratory tests, were shown to be inconsistent. All field laboratories weighed their ash correction residue to the nearest 0.1 gram, instead of to 0.01 gram, apparently because of the sensitivity of available scales. Some corrections were made on the basis of a constant factor per 100 cc. of solvent used in the test. Field laboratories also used more solvent, and the quantities of solvent varied more from test to test than those of the central laboratory.

Central laboratory and field laboratory ash corrections were compared by testing split samples taken from surface and binder mixes on 10 jobs. The results showed that the field laboratories had a smaller $\overline{X}$ ash correction and were, on the average, less variable. On individual jobs, this trend was not so pronounced, as shown by the following tabulation.

Ash correction:	*Grams*
Surface mix, average ($\overline{X}$):	
Central laboratory	4.60
Field laboratory	4.10
Binder mix, average ($\overline{X}$):	
Central laboratory	4.95
Field laboratory	4.10
Surface mix, standard deviation (σ):	
Central laboratory	2.48
Field laboratory	1.64
Binder mix, standard deviation (σ):	
Central laboratory	2.28
Field laboratory	1.84

Table 5.—Average sieve data, aggregate residue and asphalt content—from extraction tests of samples obtained from same mix at two locations on 10 projects in three States

	Average standard deviation ($\bar{\sigma}$)		Average shift of average ($\bar{X}$) from job mix formula target		Average variance components as a percent of total variance (σ_a^2)						Average percent compliance with job mix formula tolerances	
					Testing		Sampling		Material			
Sample location	Truck	Core	Truck	Core	Truck	Core	Truck	Core	Truck	Core	Truck	Core
	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
Sieve:												
3/4 in. or 1/2 in.	1.33	1.69	1.58	1.11	74	32	1	2	25	66	99	100
3/8 in.	2.34	2.42	1.16	0.87	37	39	22	14	41	47	98	98
No. 4	2.89	2.96	1.68	2.14	26	27	21	28	53	45	85	83
No. 8 or 10	2.53	2.58	1.81	2.50	19	21	13	24	68	55	84	87
No. 20 or 30	1.52	1.73	1.59	2.06	12	13	18	17	70	70	92	86
No. 40 or 50	1.45	1.66	1.80	2.00	22	16	6	8	72	76	84	79
No. 80 or 100	1.06	1.09	1.34	1.63	27	21	10	9	63	70	79	74
No. 200	0.98	0.97	1.05	1.26	27	24	11	10	62	66	74	70
Average	1.76	1.88	1.50	1.69	31	24	13	14	56	62	87	85
Asphalt	0.22	0.22	[1] 0.23	[1] 0.22	32	40	11	22	57	38	[1] 61	[1] 63

[1] From six jobs in one State only.

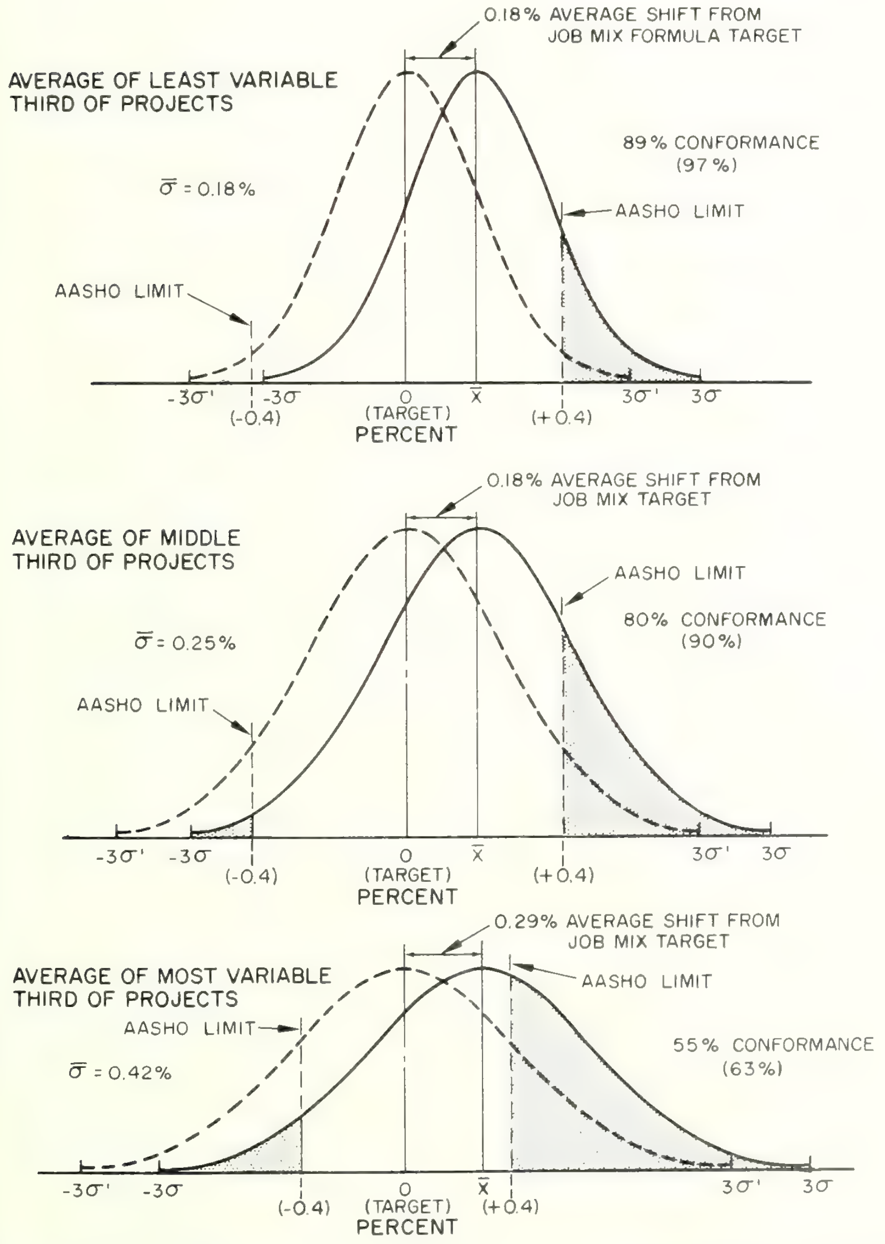

Figure 6.—Normal distribution curves, conformance to suggested AASHO guide tolerance of ±0.4% for asphalt content—three groups of surface course projects.

The lower values and less variability of the field tests do not necessarily indicate that the results are more accurate. In ash correction there is always a danger of not obtaining a true aliquot because of ash settlement in the container, which could cause indicated trends.

The variances obtained using both a constant ash correction factor and actual field correction factors were compared for binder and surface mixes using the chi square statistic, χ^2. Neither calculated value of χ^2 reached the critical 5-percent significance level. Statistically, from the Florida report (4), "it has not been demonstrated that any significant difference exists." This means both methods will produce the same results. The Florida report further states:

> "At present the evidence seems to indicate that if the operation of the extraction test could be improved (specifically: uniformity in devices, amount of solvent used, number of washes employed, speed of rotation, etc.), there is a very good possibility that the running of the ash correction as a field test could be dispensed with and a system devised using a factor assigned by the central lab., which would give statistically as good, if not better results, than are being obtained under the present system. Periodic spot checks and inspections of equipment, procedures, etc., would undoubtedly have to be made to ensure that continued high standards of operation were continuously being obtained."

Effect of extraction test equipment, operators, and laboratories

Extraction tests on the 33 projects shown in table 4 were made with either Reflux or Rotorex test equipment. Except for those of two States, all extraction and sieve tests were performed at district or central laboratories. In New Jersey (5), half the extraction tests were made at the central laboratory, and available plant testing equipment and plant inspectors were used to test the remaining half to determine whether any significant testing variability or *variability of testing variability* existed. According to the data in table 6, which is from a report by Afferton (5), testing variance, σ_t^2, for determining asphalt content was more than 15 times greater in field laboratories than in the central laboratory. The statistical test for differences of σ_t^2, using the F ratio at the 5-percent level, showed a high significance of testing variance for both courses.

A comparison test on split samples using both Reflux and Rotorex test equipment was reported in a West Virginia study (6). On the basis of t and F statistical tests, no significant differences in standard-deviation variability could be attributed to the type of test equipment. However, in companion studies, in which two sets of samples with known asphalt quantities, two operators, and both sets of equipment were used, it was shown that operator proficiency significantly affected the accuracy of the test results, possibly enough to nullify the smaller standard deviation expected of the Reflux apparatus. In another experiment, in which six operators each used Rotorex equipment to test two samples with known asphalt content (unknown to operators), the operators retained their same numerical order of proficiency.

A Florida study (4) also statistically compared field asphalt content determinations

Table 6.—Tests for significant variance difference between field and laboratory testing, 5-percent level

Test property	Testing variance (σ_t^2)		Largest variance	F ratio		Is difference significant?
	Laboratory	Field		Computed	Critical	
Top						
	Pct.	*Pct.*		*Pct.*	*Pct.*	
Asphalt content	0.0088	0.1734	Field	[1] 19.70	1.75	Yes.
Stone content	1.5500	2.9040	Field	1.87	1.75	Yes.
Sieve analysis:						
Passing 1 in., retained on ½ in.	0.7200	1.0358	Field	1.44	1.75	No.
Passing ½ in., retained on ¼ in.	2.1600	6.2827	Field	2.41	1.75	Yes.
Passing ¼ in., retained on No. 10	0.9200	0.7246	Laboratory	1.27	1.75	No.
Passing No. 10, retained on No. 30	0.9000	0.3591	Laboratory	2.51	1.75	Yes.
Passing No. 30, retained on No. 50	1.6400	1.8232	Field	1.11	1.75	No.
Passing No. 50, retained on No. 80	1.1700	0.9429	Laboratory	1.24	1.75	No.
Passing No. 80, retained on No. 200	0.7600	3.0043	Field	3.95	1.75	Yes.
Passing No. 200	0.2900	0.5121	Field	1.76	1.75	Yes.
Bottom						
Asphalt content	0.0111	0.1658	Field	[1] 14.94	1.75	Yes.
Stone content	2.9700	7.9839	Field	2.69	1.75	Yes.
Sieve analysis:						
Passing 1½ in., retained on 1 in.	13.8400	19.7247	Field	1.42	1.75	No.
Passing 1 in., retained on ½ in.	17.1100	23.1702	Field	1.35	1.75	No.
Passing ½ in., retained on ¼ in.	8.0100	9.3947	Field	1.17	1.75	No.
Passing ¼ in., retained on No. 10	3.0000	1.4930	Laboratory	2.01	1.75	Yes.
Passing No. 10, retained on No. 30	0.6200	0.3234	Laboratory	1.92	1.75	Yes.
Passing No. 30, retained on No. 50	0.5400	0.8708	Field	1.61	1.75	No.
Passing No. 50, retained on No. 80	0.4600	0.4770	Field	1.04	1.75	No.
Passing No. 80, retained on No. 200	0.8700	0.9584	Field	1.10	1.75	No.
Passing No. 200	0.3000	0.2867	Laboratory	1.05	1.75	No.

[1] Highly significant at 5-percent level.

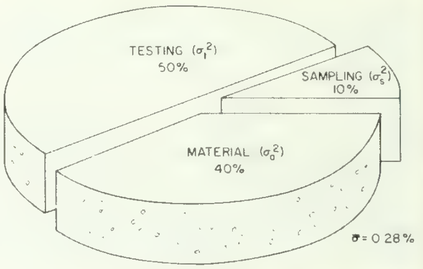

Figure 7.—Average percent of total variance, σ_o^2, attributable to testing, sampling, and material variances for asphalt content extraction tests—23 surface course projects.

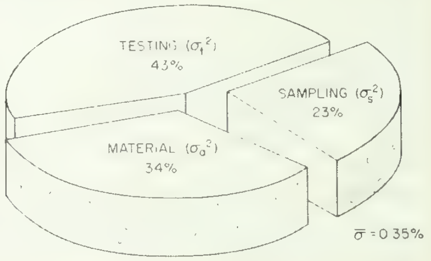

Figure 8.—Average percent of total variance, σ_o^2, attributable to testing, sampling, and material variances for asphalt content extraction tests—6 base or binder course projects.

Table 7.—Comparison of dry hot bin and extraction results [1]

Sieve size	Total percent passing			Standard deviation (σ)	
	Average hot bin	Average extraction	Average difference [2]	Pooled hot bin	Pooled extraction
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*
½ in.	99.6	99.6	0.0	0.3	0.5
⅜ in.	78.6	77.8	0.8	2.5	3.7
¼ in	47.5	46.2	1.3	3.0	2.3
No. 20	21.0	21.4	−0.4	3.7	2.3
No. 40	13.3	14.7	−1.4	3.3	1.3
No. 80	6.3	7.8	−1.5	1.9	1.3
No. 200	2.8	4.5	−1.7	1.0	1.0
Percent asphalt content		6.3			0.3

[1] Data based on 491 combined hot bin analyses and 491 extraction tests from 29 mix plants during 1962, 1963, and 1964.

[2] Difference is significant at 99 percent confidence level for all sieves except for No. 20, which is significant at 95 percent confidence level.

Table 8.—Average bituminous hot mix density data from research jobs

	Jobs	States	Average standard deviation (σ)		Average ($\bar{X}$)		Average variance components as percent of total variance (σ_o^2)			Percent compliance with State specification
			Core	Loose sample	Core	Loose sample	Testing	Sampling	Material	
Percent of theoretical density (voidless):	*Number*	*Number*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Surface	15	6	1.57		93.1		5	19	77	78
Binder	3	2	2.90		94.2		33	16	51	88
Percent of Marshall density: Surface	12	5	1.53		96.0					
Nuclear density as percent of theoretical density: surface	10	2		0.89		96.2	20	12	68	
Theoretical density (voidless):			*grams/cc.*	*grams/cc.*	*grams/cc.*	*grams/cc.*				
Surface	10	3	0.013	0.011	2.43	2.46	18	12	70	
Binder	1	1	0.029	0.013	2.48	2.48				

of binder and surface mixes made by regular plant inspectors on 10 jobs with central laboratory test results of duplicate samples and by percent of total variance from regression lines for each. Essentially the same operators were rated good-to-fair and poor-to-very poor in both types of tests, indicating that operator training and constant surveillance is necessary to achieve precise extraction test results.

Control by hot bin sieving

From 1 to 2 hours are required to complete field extraction tests now being used to determine whether bituminous hot mix conforms to the requirements of the job-mix formula. For this reason several State highway departments have been seeking quicker means to ascertain conformance so that remedial action can be taken quickly.

In a New York study (7), it was determined from research comparisons (see table 7) on dry hot bin and extraction sieve tests that dry sieving was more uniform for ½-in., ¼-in., and No. 200 sieve sizes. The extraction test yielded more consistent results for the ⅛-in. thru No. 80 sieves. As accurate printed weights of material used in each batch from each bin were obtained, it was decided to use the more rapid hot bin sieving to control the uniformity of the mix. This test was to be supplemented with a daily extraction test for aggregate passing the No. 80 and No. 200 sieves. According to the hot bin data from the 29 plants in which the tests were performed, anytime that the primary size in the coarse bin—material passing 1-in. sieve and retained on ½-in. sieve in No. 1 bin, and passing ¼-in. sieve and retained on ⅛-in. sieve in 1A bin—fell below 70 percent, the mix generally became nonuniform. By trial it was determined that a 12-percent fluctuation in this quantity from the last test was a practical limit to use in order to avoid exceeding the job-mix formula limits. On the fine aggregate bin, the same tolerance limit was applied to material retained on the No. 20 sieve, because usually about one-half of the fine aggregate was retained on this sieve.

Because of the relation of primary size to the overall conformance to the job formula, the New York State highway department is using this correlation as an indicator of uniformity. The uniformity control test is supplemented by complete hot bin analysis, usually after every fourth test. Thus, one State has been able to shift dependence on extraction test results to a secondary role.

Their inspection manual states:

"In general, production is accepted by obtaining gradation test results within the limits of a job mix formula. Hot bin analyses and uniformity tests determine the gradation of material larger than the No. 80 sieve. The extraction test is used to determine gradation of material smaller than the No. 80 sieve and also indicates the approximate bitumen content. Actual bitumen content is determined by verifying batch quantities."

Density

Permanence of bituminous pavement depends largely on the degree of compaction obtained. The compaction value is usually ex-

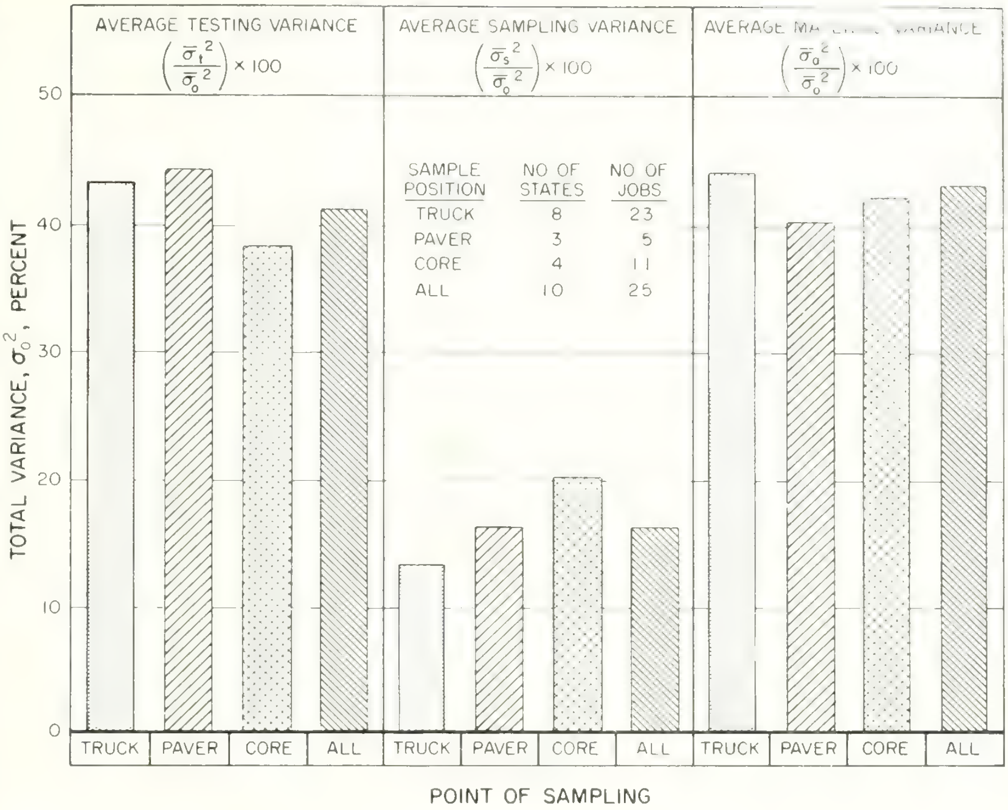

Figure 9.—Average percent of total testing variance, σ_o^2, attributable to testing, sampling, and material variances for asphalt content extraction tests—alternate sampling locations.

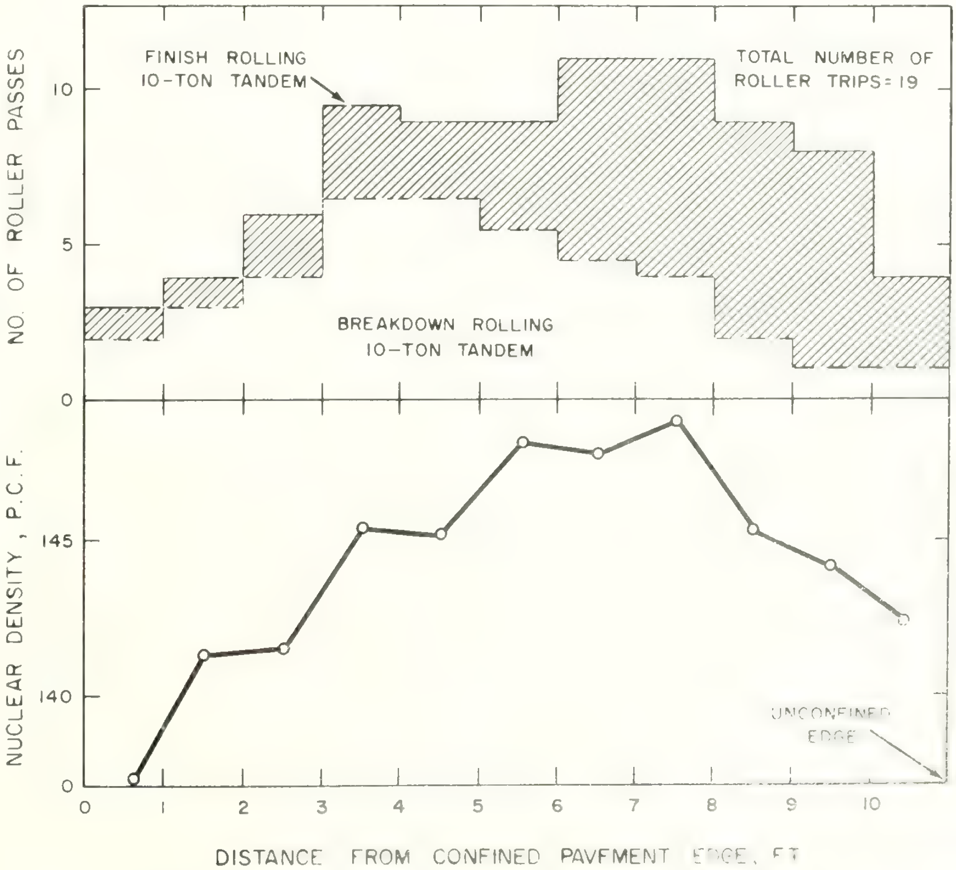

Figure 10.—Lateral variation in compactive effort and density.

pressed as a percentage of either theoretical (voidless) or Marshall density, determined by laboratory tests. Density data from several construction jobs are shown in table 8. It is quite possible that much below-specification density can be attributed to improper rolling patterns. Figure 10 was taken from a report by Kilpatrick and McQuate (8) who reported the following conclusions regarding effect of rolling pattern on density:

"Normal rolling procedures used by roller operators result in wide lateral variations in compactive effort. The number of roller passes applied in the center of the lane is usually from three to six times greater than at the lane edges.

"The lateral pattern of density is similar to the lateral pattern of compactive effort; i.e., high-in-the-middle and low-at-the-edges."

In figure 10, the density pattern across the lane approaches the shape of a normal curve. In a random selection of sample locations across the lane, sites at any distance from the edge have an equal chance of being selected.

Marshall Test Results

A number of State highway departments use the Marshall test and equipment to design the job-mix formula and control the ideal blend of aggregate, aggregate sizes, and bitumen, so that the mixture will be stable and durable when it is incorporated into the pavement. Marshall test data variations for stability, flow, and air voids from several State projects are shown in table 9. Testing and sampling variances for *stability* and *flow* values total *58* and *76* percent respectively. Variability of Marshall stability is shown in figure 11 for 3 groups of jobs: the third with the least variable standard deviation, the middle third, and the third with the most variable standard deviation. The computed $\bar{\sigma}$'s from these groups were used to plot the normal curves.

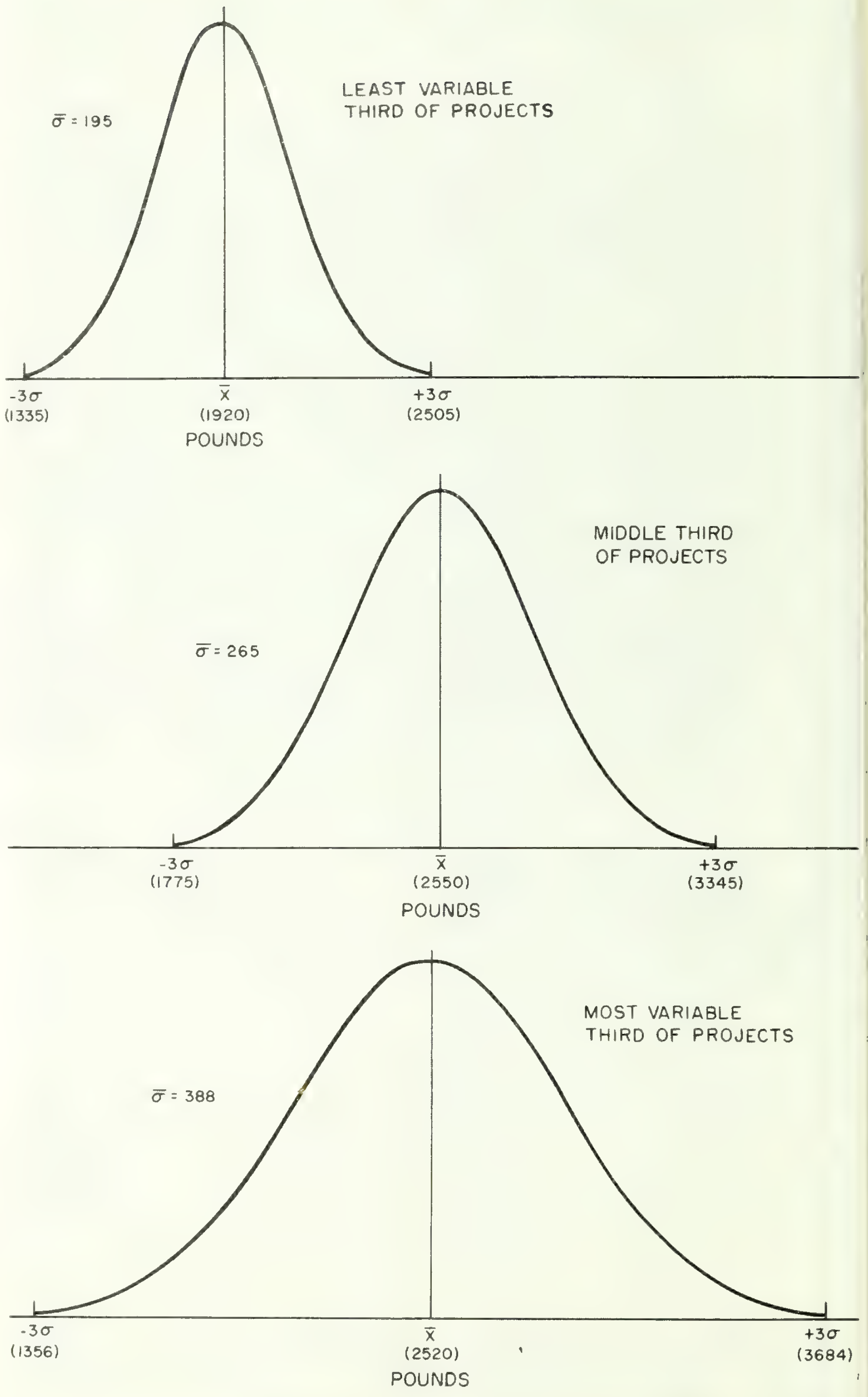

Figure 11.—Computed $3\bar{\sigma}$ limits of Marshall stability for 18 projects grouped according to size of average standard deviation, $\bar{\sigma}$.

Temperature

Another physical characteristic of the mix that may effect final density is mix temperature during breakdown rolling. Kilpatrick and McQuate (*8*) concluded that: "Breakdown rolling, both steel and pneumatic, should be completed before the pavement temperature drops below 220° F. to achieve maximum density." It is probable that final rolling, when accomplished above this critical temperature, will also produce the best results. The average standard deviation of temperature at the paver for 10 research jobs was 15° F. the range was from 6° to 22° F. Consequently, a plant producing batches with an average temperature of 275° F. will have a number of batches in the 230°–250° F. range. With temperatures in this range, it is difficult to achieve proper breakdown before the pavement cools below the reported critical 220° F.

Pavement Thickness

Thickness is another attribute needed to achieve economy of construction. A pavement [illegible] thicker than required for adequate [illegible] needlessly increases cost. A pavement [illegible] reduces service [illegible] and increases maintenance cost. Ideally, design thickness for pavements can be used to provide the most economical construction. However, research indicates that the variations in thickness of presently constructed pavements may significantly influence such performance. Data from 12 jobs in four States show that accepted surface courses have a $\bar{\sigma}$ of 0.26 inch. In other words, about 5 percent of the pavement will have a thickness over ½ inch less than desired

Table 9.—Hot mix Marshall test data variations for stability, flow, air voids

	Projects	States	Average standard deviation (σ)	Average ($\bar{X}$)	Average variance components as a percent of total variance (σ_t^2)		
					Testing	Sampling	Material
	Number	*Number*					
Marshall stability___pounds__	18	4	283	2,305	38	20	42
Marshall flow ____ 100 in. _	15	2	1.29	8.62	62	14	24
Marshall air voids______pct__	18	4	1.00	4.33	21	24	55

f the average corresponds to the specification. f concepts developed by Rex (*9*) are utilized, he computed expected service life for 5 ercent of the area of a 3-inch pavement will e only ⅔ of the design life.

Conclusions

The production of high quality bituminous avements requires the diligence of all conerned—the producer, the contractor and the ontracting agency. The statistically measured ariations (parameters) of accepted construcion presented in this article indicate that nuch more variability exists than is revealed y the usual acceptance tests. Variations in xcess of those normally expected for *good ractice* were prevalent on almost every job tudied. At present, the full significance of uch variations cannot be assessed. Large ampling and testing errors virtually prevent true evaluation of the material variation on specific job. Also, it is difficult to assess the legree to which the variations affect actual avement performance.

Because performance has not always been atisfactory, the need for improvement is bvious. Research results indicate that much mprovement could be obtained and testing oad reduced by the following changes:

- Adjust tolerance limits on gradation to onform to the principle of most tolerance on argest fraction retained on a sieve.
- Control the uniformity of gradation of the ixture by hot bin sieve tests, when a printed ecord of batch weights is available.
- Reduce to a minimum the number of ieves used for control testing.
- Exercise more diligence in the training and urveillance of operators performing control nd acceptance tests.
- Require installation of automatic features n asphalt plants and finishers to reduce uman error.
- Use random sampling to obtain all test portions.

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New Publications

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Drainage of Highway Pavements

Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, March 1969 ($1 a copy), contains procedures for estimating storm runoff from pavement areas, computing flow in gutters, and designing and spacing inlets for removing water from the pavement surface. Typical inlet designs are given and tables and charts are provided for computing flow in gutters and the water intercepted by curb opening and grate inlets.

Other hydraulic publications available from the U.S. Government Printing Office are listed on the inside back cover.

Ultrasonic Testing Inspection for Butt Welds in Highway and Railway Bridges

Ultrasonic Testing Inspection for Butt Welds in Highway and Railway Bridges (40 cents a copy) is a 49-page manual prepared by the Bethlehem Steel Corporation in cooperation with the Bureau of Public Roads. It is a training and reference manual for inspectors using special techniques develope by Bethlehem Steel Corporation for bu weld inspection. It explains the theory an operation of ultrasonic pulse-echo, flaw detection equipment and describes specia procedures for using this equipment.

The use of ultrasonics as a nondestructiv test method for weld inspection is steadil increasing, largely because it is less costl and less time consuming than the metho now being used. In recognition of thes advantages, the Bureau of Public Roads i issuing a *Specification for the Ultrason Testing of Butt Welds in Highway and Railwa Bridges* that will be applicable to steel bridge constructed under Federal-aid projects. Thi specification is to be used in conjunction wit the manual.

Third Generation Destination Signing—An Electronic Route Guidance System

(*Continued from p. 200*)

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Four level interchange near Dayton, Ohio, where highways [illegible] and U.S. 35 intersect. Th[illegible] [illegible] of four levels reduced the [illegible] of the construction area in [illegible] ostly industrial vicinity.

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The most prevalent type of accident on the Interstate Highway System is the single-vehicle accident, in which the vehicle strikes a fixed roadside object or overturns.

Fatal Accidents on Completed Sections of the Interstate Highway System, 1968

BY THE OFFICE OF
TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by HAROLD R. HOSEA,
Accident Records Analyst,
Programs Division

Introduction

REPORTS of 2,754 fatal accidents that occurred in 1968 on sections of the Interstate Highway System complying fully to Interstate design standards have been analyzed from police investigation reports obtained from the States. These accidents, which constitute slightly less than 90 percent of the total allocated to the Interstate System under statistical reporting procedures developed by the Current Planning Division, Bureau of Public Roads, are discussed here. Accidents that occurred on certain frontage, service, and connector roads were omitted, as were a few for which reports were received too late to include in the discussion. Statistical data, compiled in accordance with the Planning Division's procedures, are summarized in an annual publication of the Bureau of Public Roads (*1*)[1].

The fatal accidents that occurred on the Interstate System in 1967 were also analyzed, but no formal report was issued for general distribution. Certain data for the two years, 1967 and 1968, were compared, and important differences in accident patterns are reported here, although not all the data are readily comparable because of recent refinements in statistical treatment.

Number and Characteristics of the Accidents

Types of accidents

The 2,754 accidents were divided into two broad categories for analysis: those involving a single vehicle, and those involving two or more vehicles. As indicated in table 1, the single-vehicle accidents accounted for two-thirds of the total. This ratio is virtually identical with that for the 1967 accidents, but there were minor differences in the proportions of specific types of accidents. The most significant difference was in the number of head-on collisions, which amounted to a smaller percentage of the total in 1968 than in 1967. This was true of both principal types of head-on collisions—those caused by wrong-way drivers and those caused by out-of-control vehicles that crossed medians.

Pedestrian accidents accounted for more than 7 percent of the accidents in both years, a proportion that seems high, as pedestrians are excluded by law or regulation from the Interstate System. In the 1967 study, it was

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[1] Italic numbers in parentheses identify the references listed on page 224.

Table 1.—Fatal accidents on completed sections of the Interstate System, 1968—accident types, fatalities, injuries, and property damage

Type of accident	Accidents			Fatalities		Injuries		Property damage	
	Number	Percent		Total	Per accident	Total	Per accident	Total	Per accident
		Total	Subgroup						
								Thousands of dollars	*Dollars*
Total accidents, all types	2,754	100.0	-------	3,326	1.21	3,067	1.14	7,783.9	2,826
Single vehicle:									
Ran off road	1,462	53.1	79.4	1,685	1.16	1,223	0.84	3,281.3	2,244
Overturn on road	31	1.1	1.7	37	1.19	36	1.16	35.6	1,148
Collision with parked vehicle	96	3.5	5.2	114	1.19	111	1.16	440.2	4,585
Pedestrian:									
Persons outside their vehicles	61	2.2	3.3	65	1.07	11	0.18	15.3	251
Trespassers	153	5.6	8.3	154	1.01	12	0.08	40.0	261
Total pedestrian	214	7.8	11.6	219	1.02	23	0.11	55.3	258
Other	[1] 39	1.4	2.1	42	1.08	30	0.77	75.2	1,928
Total single vehicle	1,842	66.9	100.0	2,097	1.14	1,423	0.77	3,887.6	2,111
Multiple vehicle:									
Rear-end collision	411	14.9	45.1	504	1.23	667	1.62	2,006.6	4,882
Head-on collision:									
Wrong-way driver	131	4.8	14.4	230	1.76	222	1.69	416.6	3,180
Vehicles from opposing lanes	164	5.9	18.0	243	1.48	417	2.54	783.3	4,776
Other	14	0.5	1.5	22	1.57	25	1.79	57.5	4,107
Total head-on collision	309	11.2	33.9	495	1.61	664	2.15	1,257.4	4,069
Broadside collision	65	2.4	7.1	81	1.25	129	1.98	197.8	3,043
Sideswipe	127	4.6	13.9	149	1.17	184	1.45	434.5	3,421
Total multiple vehicle	912	33.1	100.0	1,229	1.35	1,644	1.80	3,896.3	4,272

[1] Primarily, vehicles that struck other objects or non-motor vehicles on the road, and accidents in which occupants fell from vehicles.

assumed that relatively few of the pedestrians were *trespassers* as distinguished from persons who had left their vehicles for one reason or another. To test this assumption, the two groups were coded separately in the study reported here. Of the 219 pedestrian fatalities, 154, or 70 percent, were actually *trespassers.* (See table 1.)

Collisions with properly parked vehicles are classified as single-vehicle accidents. Collisions with vehicles standing on the traveled way are classified as other types of accidents, depending on the position of the vehicle struck. In both years, accidents in which vehicles ran off the road and did not collide with another vehicle were the most frequent. This type of accident is discussed later in detail.

Fatalities and injuries

The 2,754 accidents resulted in 3,326 fatalities or 1.21 deaths per accident. Single-vehicle accidents resulted in an average of 1.14 fatalities per accident, which corresponded to 1.35 fatalities per accident in multiple-vehicle accidents. These rates did not differ significantly from those for 1967. Head-on collisions caused by wrong-way drivers produced the highest fatality rate, 1.76 per accident.

There were 1.14 nonfatal injuries per fatal accident reported in 1968, a somewhat lower average than that for 1967. Total injuries in all fatal accidents were 3,067.

Accident costs

To estimate the economic loss in fatal accidents, the following figures suggested by the National Safety Council *(2)* have been used for fatalities and injuries. It is recognized that there are numerous other figures widely used for this purpose and any different values can be substituted by simple multiplications.

Table 2.—Fatal accidents on completed sections of the Interstate System, 1968—accident types and light conditions

Type of accident	Total	Light condition					
		Daylight	Darkness			Dawn or dusk	Not reported
			Total	Unlighted	Lighted		
Total accidents, all types:							
Number	2,754	1,173	1,453	1,177	276	113	15
Percent	100	43	53	43	10	4	(1)
Single vehicle:							
Number	1,842	777	982	792	190	74	9
Percent	100	43	53	43	10	4	(1)
Ran off road:							
Number	1,462	646	746	593	153	67	3
Percent	100	44	51	41	10	5	(1)
Pedestrian:							
Number	214	58	152	123	29	2	2
Percent	100	27	72	58	14	1	(1)
Multiple vehicle:							
Number	912	396	471	385	86	39	6
Percent	100	44	52	43	9	4	(1)
Rear-end collisions:							
Number	411	139	249	205	44	20	3
Percent	100	34	61	50	11	5	(1)
Head-on collisions:							
Number	309	153	138	115	23	15	3
Percent	100	50	45	38	7	5	(1)
Wrong-way driver:							
Number	131	38	89	73	16	4	---------
Percent	100	29	68	56	12	3	---------
Vehicles from opposing lanes:							
Number	164	110	42	35	7	10	2
Percent	100	68	26	22	4	6	(1)

[1] Accidents not reported are excluded from percentage distributions.

Table 3.—Fatal accidents on completed sections of the Interstate System, 1968—vehicle types and total travel [1]

Type of vehicle	Vehicle miles [2]		Accidents		Fatalities		Injuries	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent
	Million							
Total, all types	111,368	100.0	2,754	100.0	3,326	100.0	3,067	100.0
Passenger vehicles	88,784	79.7	2,243	81.4	2,745	82.6	2,649	86.4
Property-carrying vehicles:								
Combinations	11,399	10.2	253	9.2	291	8.7	205	6.7
All single-unit trucks:								
Panels and pickups	7,563	6.8	181	6.6	---------	---------	---------	---------
Other single units	3,622	3.3	77	2.8	---------	---------	---------	---------
Total single-unit trucks	11,185	10.1	258	9.4	290	8.7	213	6.9
Total property-carrying vehicles	22,584	20.3	511	18.6	581	17.4	418	13.6

[1] Includes the one vehicle primarily responsible for each accident, as indicated by police investigation reports.
[2] Estimates (1967) by Public Roads Office of Planning.

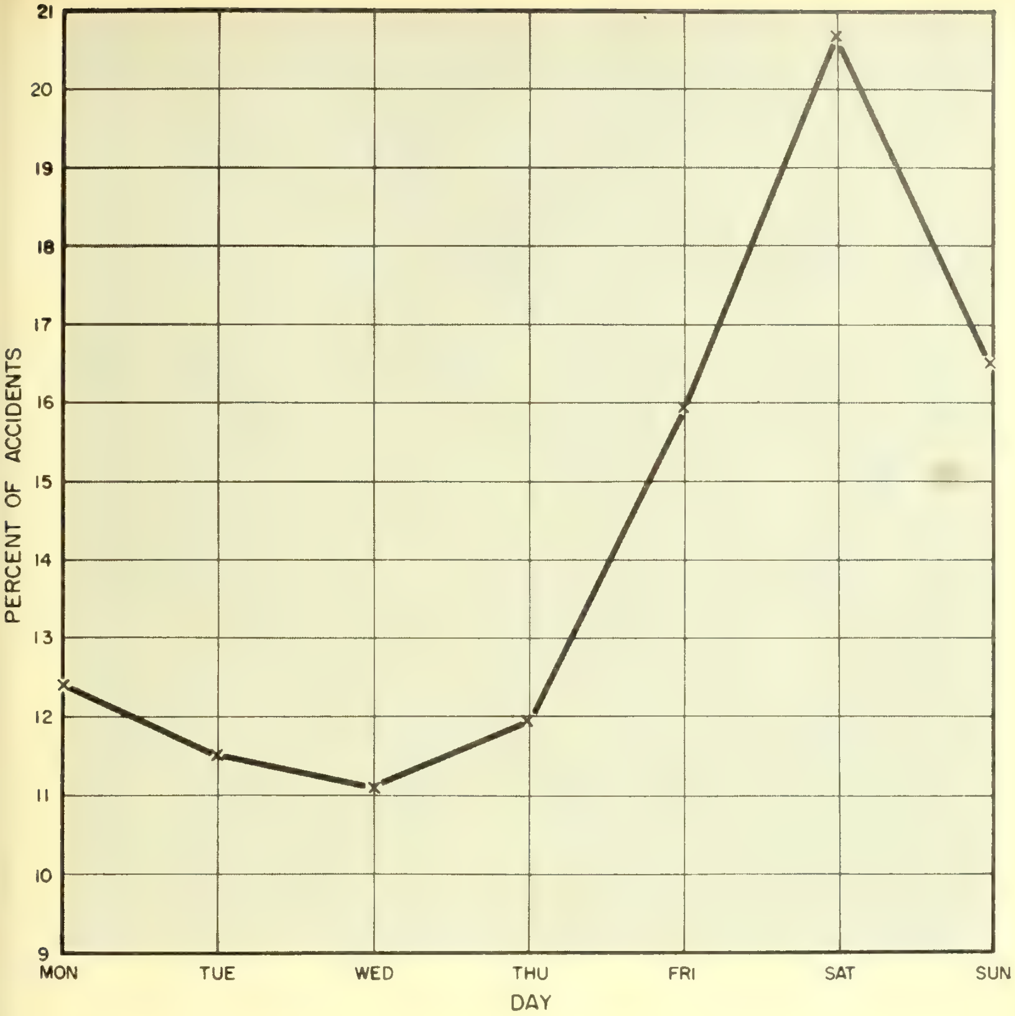

Figure 1.—Fatal accidents on the Interstate System, 1968—day of occurrence.

Estimated economic loss in fatal accidents:

	Loss
3,326 fatalities at $38,500	$128, 051, 000
3,067 nonfatal injuries at $1,377 (weighted on the basis of the three degrees of severity of injury in general use)	4, 223, 259
Property damage (see text below)	7, 783, 900
Total	140, 058, 159
Per accident	50, 856

The estimate for property damage includes both the cost of vehicle damage and the cost of replacing highway appurtenances such as guardrails, signs, light poles, etc. Typically, police investigation reports contained cost estimates of repairing the damaged vehicles, but when the estimates exceeded current retail values of the vehicles (*3*), the amounts were reduced to the latter figures. Also, the values of commercial-vehicle cargoes damaged in accidents were often inadequate for precise estimates of property damage.

Day and Time of Occurrence

The distribution of fatal accidents by day of the week is shown in figure 1. The maximum frequency occurred on Saturday when 20.7 percent of the accidents were recorded. The low point was on Wednesday when 11.1 percent of the accidents occurred. Over half, 53.1 percent, of the accidents occurred on weekends, Friday through Sunday.

The highest percentage of single-vehicle accidents for any interval occurred between 2 and 3 a.m. (fig. 2). The peak for multiple-vehicle accidents was 3 hours earlier, beginning at 11 p.m. Of the total accidents, 6.4 percent occurred in the 2 a.m. hour. The proportion for single-vehicle, off-the-road accidents was slightly higher—7.0 percent. The smallest percentages occurred in the 9 a.m. and 11 a.m. hours for multiple- and single-vehicle accidents, respectively.

Accident Environment

Light conditions

More than half the accidents, 53 percent, occurred at night and only a fifth of these were on lighted sections of highway. (See table 2.) This might be expected because lighting on the Interstate System is largely limited to urban sections and interchanges. A higher percentage, 72 percent, of the pedestrian accidents occurred at night, and although precise information was not available, numerous investigation reports cited dark clothing and intoxication as factors in these accidents. Also, collisions caused by wrong-way drivers occurred more frequently during hours of darkness, although the opposite was true in collisions caused by vehicles that crossed medians.

Weather and pavement conditions

Four-fifths of the accidents occurred during clear or cloudy weather, 15 percent in rainy weather, 3 percent during snow, and 2 percent in fog. There were no significant deviations from this pattern for the different types of accidents except rear-end collisions, 5 percent of which occurred in fog. Investigation reports noted a few accidents in which strong wind was a factor in off-the-road accidents, principally those involving small foreign cars.

As might be expected, the pattern of accidents related to pavement conditions was essentially similar to that related to weather conditions. About a fifth of the accidents occurred on wet pavements, and 4 percent on ice or snow. The outstanding exception was that more than half, 52.2 percent, of the head-on collisions that resulted from out-of-control vehicles crossing medians happened on wet or icy pavements. Somewhat unexpected was the fact that fewer than a fourth of the single-vehicle, off-the-road accidents occurred on slippery roads.

Highway alinement

More than three-fourths, 78 percent, of the fatal accidents took place on straight sections of highway, and the remainder were nearly equally divided between right and left curves. Single-vehicle, off-the-road accidents occurred more frequently on curved sections, 29 percent of the total, and again they were nearly equally divided between right and left curves. A third of the accidents were on grades—two-thirds on straight sections, and the remainder on curves. In the several individual types of accidents there were no major deviations from these patterns except in rear-end and head-on collisions, nine-tenths of which were on straight sections.

Vehicles

Vehicle types

To assess the importance of different types of vehicles in these accidents, it was necessary to assign the primary *responsibility* for each accident to an individual vehicle.[2] Although an element of judgment was involved, significant errors appeared minimal. In two-thirds of the accidents, only one moving vehicle was involved (see table 1). Eleven percent of the accidents were head-on collisions resulting

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[2] *Responsibility* does not necessarily imply violations by the drivers involved.

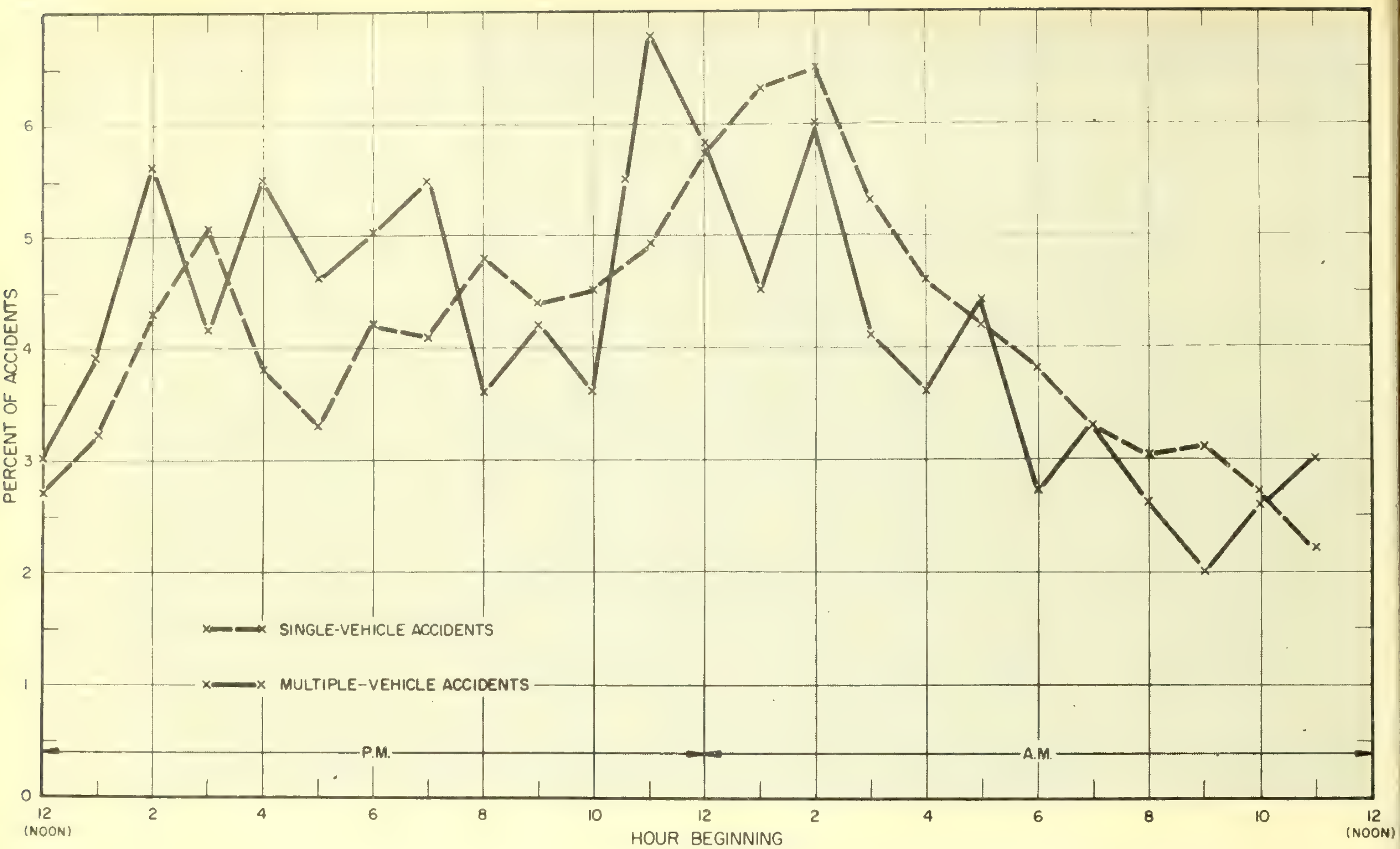

Figure 2.—Fatal accidents on the Interstate System, 1968—hour of occurrence.

from wrong-way drivers or out-of-control vehicles from opposing lanes—accidents in which there is little or no question of responsibility. Rear-end collisions are usually, but not always, the responsibility of the striking vehicle. In these accidents, as well as in broadside collisions and sideswipes, responsibility was assessed on the basis of the details of investigation reports, including the narratives, diagrams, notations of violations, and related data.

The volume of travel and the number of accidents, fatalities, and injuries for each of the principal types of vehicles primarily responsible for the accidents are shown in table 3. Accident responsibility was determined from the investigation reports, as outlined in the preceding paragraph. Passenger vehicles are shown as a single group because the estimates of vehicle-miles of travel were not available for individual types. The percentages of travel, accidents, and injuries for the several types of vehicles agree closely. In general, the proportions of accidents for property-carrying vehicles are slightly less than the corresponding percentages for vehicle miles of travel. The differences for fatalities and injuries are slightly larger, probably a result of the lower average human occupany of cargo vehicles.

Elaboration of this aspect of the accident pattern is given in tables 4 and 5. The relative importance of each vehicle type in each prin-

Table 4.—Fatal accidents on completed sections of the Interstate System, 1968—types of vehicles involved in each type of accident [1]

Type of vehicle [2]	Total accidents	Single-vehicle accidents [3]				Multiple-vehicle accidents [3]				
		Total	Ran off road	Pedestrian	Collision with parked vehicle	Total	Rear-end collisions	Head-on collisions		
								Total	Wrongway	Vehicle from opposite lane
Total, all types ....number	2,754	1,842	1,462	214	96	912	411	309	131	164
Percent distribution										
Passenger										
Sedans	66.8	67.9	69.1	69.6	59.4	64.6	56.0	77.0	79.5	76.2
Convertibles	5.6	7.0	8.0	2.3	6.3	2.7	3.2	2.9	2.3	3.7
Station wagons	7.0	7.4	7.8	4.7	4.2	6.4	6.4	6.1	3.8	7.9
Motorcycles	1.1	1.0	0.8	-	1.0	1.3	1.9	------	------	------
Other [4]	0.9	0.8	0.7	1.0	1.0	1.1	1.1	0.7	0.7	0.6
Total passenger vehicles	81.4	84.1	86.4	77.6	71.9	76.1	68.6	86.7	86.3	88.4
Property carrying:										
Combinations	9.2	7.5	6.5	9.4	19.8	12.5	19.5	2.9	------	4.9
Panel and pickups	6.6	5.8	5.7	6.5	3.1	8.2	7.5	9.1	13.0	4.9
Single unit trucks [5]	2.8	2.6	1.4	6.5	5.2	3.2	4.4	1.3	0.7	1.8
Total property carrying	18.6	15.9	13.6	22.4	28.1	23.9	31.4	13.3	13.7	11.6

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[1] This table is the converse of table 5.

[2] Type of vehicle primarily responsible for each accident, as indicated by police investigation reports.

[3] Types of accidents that occurred less frequently (see table 1) not shown separately.

[4] Includes eight small (less than 10 passenger) buses, seven large buses, one school bus, and eight campers.

[5] Includes a few highway maintenance vehicles.

Table 5.—Fatal accidents on completed sections of the Interstate System, 1968—types of accidents in which each vehicle type was involved[1]

Type of accident	Passenger vehicles[2]						Property-carrying vehicles			
	Total	Sedans	Convertibles	Station wagons	Motorcycles	Other[3]	Total	Combinations	Panels and pickups	Single-unit truck
Total, all types ... number	2,243	1,839	155	194	31	24	511	253	181	77
Percent distribution:										
Single vehicle:										
Ran off road	56.3	54.9	75.5	59.3	35.5	41.7	38.9	37.5	46.4	25.9
Overturn on road	1.1	0.9	-------	1.5	19.4	-------	1.2	(4)	0.6	5.2
Collision—parked vehicle	3.1	3.1	3.9	2.1	3.2	4.2	5.3	7.5	1.6	6.5
Pedestrian	7.4	8.1	3.2	5.1	-------	8.2	9.4	7.9	7.8	18.2
Other	1.2	1.0	1.3	2.1	3.2	4.2	2.5	1.6	2.2	6.5
Total single vehicle	69.1	68.0	83.9	70.1	61.3	58.3	57.3	54.9	58.6	62.3
Multiple vehicle:										
Rear-end collision	12.6	12.5	8.4	13.4	25.8	20.8	25.2	31.6	17.2	23.4
Head-on collision:										
Wrong-way driver	5.0	5.7	1.9	2.6		4.2	3.5	-------	9.4	1.3
Vehicle from opposite lane	6.5	6.8	3.9	6.7	-------	4.2	3.7	3.2	4.4	3.9
Other	(4)	(4)	-------	0.5	-------	-------	0.7	(4)	1.6	-------
Total head-on collision	11.9	12.9	5.8	9.8	-------	8.3	8.0	3.6	15.4	5.2
Broadside collision	2.5	2.5	0.6	3.1	3.2	12.5	1.6	1.6	1.6	1.3
Sideswipe	3.9	4.1	1.3	3.6	9.7	-------	8.0	8.3	7.2	7.8
Total multiple vehicle	30.9	32.0	16.1	29.9	38.7	41.7	42.7	45.1	41.4	37.7

[1] This table is the converse of table 4.

[2] Includes the one vehicle primarily responsible for each accident, as indicated by police investigation reports.

[3] Includes eight small (less than 10 passenger) buses, seven large buses, one school bus, and eight campers.

[4] Less than 0.5 percent.

Table 6.—Age and sex of drivers in fatal accidents on completed sections of the Interstate System, 1968[1]

Age	Total accidents			Single-vehicle accidents			Multiple-vehicle accidents		
	Male	Female	Total	Male	Female	Total	Male	Female	Total
Drivers, all ages[2] ... number	2,315	411	2,726	1,533	287	1,820	782	124	906
Percent distribution:									
Under 25:									
15 and under	0.5	-------	0.4	0.5	-------	0.4	0.5	-------	0.3
16	0.9	0.7	0.9	1.1	1.1	1.1	0.5	-------	0.4
17	1.3	1.5	1.4	1.5	1.7	1.5	1.0	0.8	1.0
18	4.2	4.9	4.3	5.0	4.2	4.9	2.5	6.5	3.1
19	4.4	2.2	4.0	5.2	2.1	4.7	2.6	2.4	2.6
20–24	22.4	19.2	21.9	24.5	21.6	24.1	18.3	13.7	17.7
Total under 25	33.7	28.5	32.9	37.8	30.7	36.7	25.4	23.4	25.1
25–34	22.1	21.9	22.0	21.5	25.1	22.0	23.3	14.5	22.1
35–44	17.8	21.9	18.4	17.6	20.5	18.0	18.3	25.0	19.2
45–54	13.8	11.9	13.5	12.3	9.4	11.9	16.8	17.7	16.9
55–64	7.5	9.2	7.8	6.3	9.8	6.9	9.8	8.1	9.6
65–74	3.7	5.6	4.1	3.3	3.8	3.3	4.7	9.7	5.4
75 and over	1.4	1.0	1.3	1.2	0.7	1.2	1.7	1.6	1.7

[1] Data in this table refer to the one driver primarily responsible for each accident, as indicated by police investigation reports.

[2] Excludes 28 drivers whose ages were not reported.

cipal type of accident is shown in table 4.[3] Of the vehicles involved in off-the-road accidents, 86.4 percent were passenger cars. The corresponding percentage for tractor-trailer combinations was only 6.5. Wrong-way drivers of passenger vehicles caused 86.3 percent of the head-on collisions of this type, whereas wrong-way drivers of panels and pickups caused only 13.0 percent. Cargo vehicles, however, were involved more frequently in read-end collisions.

The distribution of accident types in which each vehicle category was involved is shown in table 5, which is the inverse of the relation shown in table 4. In single-vehicle accidents, 69.1 percent of the passenger vehicles and 57.3 percent of the property-carrying vehicles were involved. Compared to 31.6 percent of the tractor-trailer combinations, only 12.6 percent of the passenger cars were involved in rear-end collisions.

Table 7.—Fatal accidents on completed sections of the Interstate System, 1968—condition of drivers, as reported by investigators[1]

Driver condition	Total accidents	Single-vehicle accidents			Multiple-vehicle accidents	
		Total	Ran off road	Collision with parked vehicle	Total	Rear end
Total drivers, all conditions:						
Number	2,754	1,842	1,462	96	912	411
Percent	100	100	100	100	100	100
Condition not reported:						
Number	684	445	391	21	239	97
Percent	25	24	27	22	26	24
Condition reported:						
Number	2,070	1,397	1,071	75	673	314
Percent	100	100	100	100	100	100
Normal:						
Number	1,504	933	648	44	571	249
Percent	73	67	61	59	85	79
Total defects reported:						
Number	566	464	423	31	102	65
Percent	100	100	100	100	100	100
Asleep:						
Number	433	368	342	20	65	46
Percent	77	79	81	65	64	71
Fatigued:						
Number	52	32	25	6	20	14
Percent	9	7	6	19	19	22
Ill:						
Number	46	35	32	3	11	4
Percent	8	8	7	10	11	6
Other:						
Number	35	29	24	2	6	1
Percent	6	6	6	6	6	2

[1] Data in this table refer to the one driver responsible for each accident, as indicated by police investigation reports

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[3] Vehicles deemed primarily responsible for each of the accidents.

Table 8.—Fatal accidents on completed sections of the Interstate System, 1968—sobriety of drivers, as reported by investigators [1]

Driver sobriety	All accidents	Single-vehicle accidents			Multiple-vehicle accidents			
						Head-on collisions		
		Total	Ran off road	Collision with parked vehicle	Total	Total	Wrong-way drivers	Rear-end collisions
Total drivers, all conditions:								
Number	2,754	1,842	1,462	96	912	309	131	411
Percent	100	100	100	100	100	100	100	100
Sobriety not reported:								
Number	723	493	445	24	224	85	38	88
Percent	26	27	30	25	25	28	29	21
Sobriety reported:								
Number	2,031	1,349	1,017	72	688	224	93	323
Percent	100	100	100	100	100	100	100	100
Not drinking:								
Number	1,382	931	660	49	451	121	24	230
Percent	68	69	65	68	66	54	26	71
Had been drinking:								
Number	649	418	357	23	237	103	69	93
Percent	32	31	35	32	34	46	74	29
Intoxicated:								
Number	187	101	84	8	86	42	31	28
Percent	9	7	8	11	13	19	33	9
Impaired:								
Number	70	41	34	3	29	9	6	10
Percent	4	3	3	4	4	4	7	3
Not impaired:								
Number	29	27	21	1	2	1	1	1
Percent	1	2	2	2	(2)	(2)	1	(2)
Extent of impairment not reported:								
Number	363	249	218	11	120	51	31	54
Percent	18	19	22	15	17	23	33	17

[1] Data in this table refer to the one driver responsible for each accident, as indicated by police investigation reports.
[2] Less than 0.5 percent.

Table 9.—Fatal accidents on completed sections of the Interstate System, 1968—age and sobriety of drivers, as reported by investigators [1]

Age	Total number of drivers	Drivers not reported		Total number reported	Reported as drinking		
		Number	Percent of all drivers		Number	Percent of all drivers	Percent of reported drivers
Number of drivers, all ages	3,972	807	20.3	3,165	700	17.6	22.1
Under 25:							
17 and under	93	13	14.0	80	8	8.6	10.0
18	146	31	21.2	115	29	19.9	25.2
19	137	31	22.6	106	32	23.4	30.2
20-24	742	166	22.4	576	175	23.6	30.4
Total under 25	1,118	241	21.6	877	244	21.8	27.8
25-34	925	149	16.1	776	177	19.1	22.8
35-44	789	146	18.5	643	144	18.3	22.4
45-54	565	129	22.8	436	91	16.1	20.9
55-64	336	70	20.8	266	28	8.3	10.5
65-74	143	28	19.6	115	11	7.7	9.6
75 and over	42	11	26.2	31	2	4.8	6.5
Not reported	54	33	61.1	21	3	5.6	14.2

[1] This table includes drivers of all vehicles involved in the accidents as opposed to table 8, which includes only one driver for each accident.

Vehicle defects

Police investigation reports gave little information on the condition of vehicles involved in accidents. Some of the report forms had no provision for information on vehicle condition. Many times this information was only one of several items listed in a general category for reporting circumstances that apparently contributed to the accident. The other items referred to driver condition, road defects, excessive speed, weather conditions, etc. Seldom was more than one item checked, and often none was checked, except when an obvious violation was involved.

Information on vehicle condition usually is lacking in these reports because the immediate and urgent responsibility of investigating officers is to injured persons and to restoration of traffic, leaving little or no opportunity for detailed inspection of the vehicles on the scene. Consequently, only 10 percent of the reports referred to apparent vehicle defects and nearly three-fourths of these referred to tires—usually to inadequate tread depth rather than to actual tire failures. A very few reports listed defective brakes or lights. The proportion of vehicles reported to have defects was essentially the same as that in the 1967 study, and was not inconsistent with a National Safety Council finding that defective vehicles were involved in 7 percent of the fatal accidents on turnpikes in 1966 (*4*).

Seat belts

Information on seat-belt availability and use was reported for three-fifths of the drivers involved in the 2,754 fatal accidents. Although belts were available to 45 percent of these drivers, only half used them. There was no significant difference between the number of male and female drivers who used belts. The foregoing percentages are affected by the number of cargo vehicles, about a fifth of the total, relatively few of which were equipped with restraining devices.

Vehicle Drivers

Age and sex of drivers

More than four-fifths of the drivers primarily responsible for the fatal accidents[4] were males. According to Public Roads estimates (*5*), males constituted 58.5 percent of all licensed drivers, but any comparison of this figure with accident involvement is misleading because relative exposure was not considered. Also, males were overrepresented, as few cargo vehicles were driven by females. There were no significant differences between the proportions of males and females involved in the several types of accidents.

Compared with 28.5 percent of the females, slightly more than a third of the male drivers were less than 25 years old (table 6). In this age group, more drivers, both male and female, were involved in single-vehicle accidents than in multiple-vehicle accidents. According to Public Roads estimates, approximately 25 percent of all licensed drivers are in the under-25 age group, compared with the 32.9 percent involved in these accidents. Again, this comparison is deceptive as it ignores relative exposure. Younger drivers are almost certainly overrepresented in total traffic volumes, although no statistical documentation exists to support this fact. Conversely, drivers 65 years old and older constitute nearly 8 percent of the licensed drivers, but these drivers were involved in only 5.4 percent of the accidents.

There were some significant differences in the age distributions of drivers in specific types of accidents. Drivers under 25 years of age, for example, were responsible for almost 40 percent of the single-vehicle, off-the-road accidents and for only 25.1 percent of the multiple-vehicle accidents. Drivers 65 years old or older caused 15 percent of the head-on collisions, which resulted from driving in wrong lanes of divided highways, but they were involved in fewer than 5 percent of the single-vehicle accidents.

Physical condition of drivers

The physical condition of three-fourths of the drivers responsible for the 2,754 accidents was reported by investigating officers (table 7). No physical defect was recorded in 73 percent of the reports that contained this information. The condition most frequently reported was sleep or dozing—77 percent of the defects recorded for all accidents and

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[4] Primary responsibility for drivers was determined by the procedure outlined under *Vehicles*. Undoubtedly, in some pedestrian fatalities, victims rather than drivers are primarily responsible.

Table 10.—Characteristics of single-vehicle, off-the-road fatal accidents on completed sections of the Interstate System, 1968

Type of accident	Total		Vehicles leaving the road			
			Left side of road		Right side of road	
	Number	Percent	Number	Percent	Number	Percent
Total accidents, all types	1,462	100.0	695	100.0	767	100.0
Struck fixed object:						
Total	1,208	82.6	540	77.7	668	87.1
Overturned	480	32.8	230	33.1	250	32.6
Overturned only	245	16.8	152	21.9	93	12.1
All overturns	725	49.6	382	55.0	343	44.7
Off the road only	9	0.6	3	0.4	6	0.8

81 percent for drivers involved in off-the-road accidents. Presumably, there is no clear line of demarcation between the drivers who were asleep or dozing and those who were fatigued. Fatigue constituted an additional 9 percent of the driver defects reported. Particularly significant is the fact that 92 percent of the drivers responsible for rear-end collisions who were reported as having defects were asleep or fatigued. The percentage for collisions with parked vehicles was slightly smaller. Eight percent of the defects reported involved illness; most frequently mentioned were cardiac conditions and effects of medication. Most of the remainder, 6 percent, were drivers with bodily handicaps and those who were distracted, principally by events within the vehicles.

Sobriety of drivers

Police investigation reports, completed shortly after accidents occur, are not conclusive sources of information on alcohol in accidents. The available reports indicated that 23.6 percent of the drivers responsible for the accidents had been drinking. As shown in table 8, this percentage is equivalent to 32 percent of the drivers whose sobriety was reported, which is well below the proportion referred to in many individual sample studies in which alcohol is reported to be a contributing factor in at least half the fatal motor-vehicle accidents. There are several possible explanations for this apparent inconsistency:

- Sobriety was not reported for 26 percent of the drivers primarily responsible for the accidents discussed in this report.
- Results of blood alcohol tests, if given, were not available at the time the accident reports were completed.

Table 11.—Fixed objects struck first in single-vehicle, off-the-road fatal accidents on completed sections of the Interstate System, 1968

First object struck	Number	Percent
Total, all objects	1,208	100.0
Guardrail [1]	364	30.1
Bridge or overpass	217	18.0
Sign	97	8.0
Embankment	86	7.1
Curb	72	6.0
Divider [2]	71	5.9
Pole [3]	63	5.2
Ditch or drain	57	4.7
Culvert	51	4.2
Fence [4]	28	2.3
Tree	26	2.2
Other	76	6.3

[1] Includes cable type.
[2] Includes rail, concrete, and chainlink.
[3] Principally light poles.
[4] Principally right-of-way fences.

- Administration of tests was refused by coroners or hospitals.
- Insufficient evidence for prosecution.

Undoubtedly, the lack of information was due, in part, to inadequate investigating and reporting, as was evident from the proportions of drivers, from zero to 75 percent, whose sobriety was not reported in the several States. A tabulation was made of data from 12 States in which the sobriety of at least 90 percent of the drivers was reported. The reports from these 12 States were the most complete and detailed of available reports and were not concentrated in any section of the country. The result showed that, of the 1,276 drivers whose sobriety was reported, 22.8 percent had been drinking, a figure slightly under the 23.6 percent reported for all States.

The data shown in table 8 may, therefore, be reasonably representative of the actual situation. As indicated in the table, 32 percent of the drivers whose sobriety was reported had been drinking and 9 percent were obviously intoxicated. There was a relatively large proportion, 18 percent, whose extent of impairment was not reported because no sobriety test was administered or tests results were unavailable when initial reports were completed. More than half, 52.5 percent, of all wrong-way drivers, or 74 percent of those whose condition was reported, had consumed some alcohol. With respect to other individual types of accidents, there were no significant deviations from the general pattern.

According to a tabulation of the reported sobriety of all drivers involved in the accidents, the highest proportion of drinking drivers was in the 20–24 age group (table 9), which agrees closely with the age distribution of all male drivers responsible for the accidents (table 6). The highest proportion of drinking female drivers however was in the 25-34 age group.

None of the studies on the role of alcohol in fatal accidents reviewed was specifically concerned with the Interstate System. There are certain traffic characteristics of the Interstate System which differ from those on most other highways. In 1967, the fatal accident rate on the Interstate System was slightly more than half the rate for all other highways. Presumably this was largely a reflection of the superior design of the Interstate System. But factors other than highway design seem to be involved. Most trips on the Interstate System are probably longer than those on other highways and, except in highly urbanized areas, they tend to be for somewhat different purposes. Travel to and from social functions and places of entertainment where alcoholic beverages are served may be less common on the Interstate System. Moreover, alcoholic beverages, obtainable from taverns and bars located at frequent intervals on other types of highways, are not obtainable on Interstate highways.

Accordingly, there may be a lower rate of drinking and driving on the Interstate System than is suggested in certain studies usually used as a basis for conclusions concerning drinking and driving.

Table 12.—First and second fixed objects struck in single-vehicle, off-the-road fatal accidents on completed sections of the Interstate System, 1968

First object struck	Guardrail	Bridge or overpass	Sign	Embankment	Curb	Divider	Pole	Ditch	Culvert	Fence	Tree	Other
Second object struck:												
None	151	181	76	58	16	47	44	46	38	9	21	55
Bridge	102	16	6	2	10	2	4	1	2	1	1	3
Embankment	36	4	2	2	4	1	2	5	2	5		1
Guardrail	18	4	2	1	17	10	3		2			3
Pole	17	3	3	1	11	2	1	1		3		1
Sign	14				4	5	2					6
Ditch	6	4	3		3	1	4	1	1	5	1	1
Culvert	3		1	4	1			2		1		1
Curb	3	1		1	2							
Divider	3	1	1	2		1	1					1
Fence	4	1	1	6	1	2	2		1			
Tree	2			5	2			1	3	3	3	4
Other	5	2	2	4	1				2	1		1
Total, all collisions	364	217	97	86	72	71	63	57	51	28	26	76

Single-Vehicle, Off-the-Road Accidents

More than half, 53.1 percent, of the 2,754 accidents involved vehicles that ran off the road and did not collide with another vehicle; they constituted almost four-fifths of the single-vehicle accidents. A summary of the characteristics of these accidents is given in table 10. More than four-fifths of the vehicles struck fixed objects after leaving the road and two-fifths of these subsequently overturned. Another 16.8 percent overturned but did not strike a fixed object—this does not include the 31 vehicles which overturned without leaving the road. Overturns occurred in half the accidents of this type. Fewer than one percent of these off-the-road vehicles neither struck a fixed object nor overturned; in several of these accidents the fatalities resulted from ejection of the occupants.

Of all the vehicles that left the road, slightly more than half went off the right side. These vehicles struck fixed objects more frequently, as more fixed objects are generally placed on the right side of the road than in medians. The placement of fewer objects in medians presumably is the reason for the larger proportion of vehicles that ran off to the left and overturned without striking a fixed object.

The frequency with which different types of fixed objects were struck first by vehicles involved in these accidents is shown in table 11. Guardrails ranked first, 30 percent of the total, because they are usually the closest targets for out-of-control vehicles. Dividers are shown separately despite the similarity of their function to that of guardrails.

Of the 1,208 vehicles that left the road and struck a fixed object, 468, or nearly two-fifths of the total, subsequently struck some other type of fixed object. For example, 102 of the vehicles that first struck a guardrail subsequently hit a bridge or overpass element. (See table 12.) Where initial impacts involved bridge or overpass elements, relatively few vehicles subsequently struck other objects, as might be expected because of the substantiality of bridge and overpass structures. Virtually none of the reports distinguished the fixed-base signs and poles struck from those that have breakaway features.

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(*3*) *Official Used Car Guide*, current issue, National Automobile Dealers Used Car Guide Co., Wash., D. C.

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PB 173948	Splice [illegible] August 1965.
PB 173949	A [illegible] Bridge Decks, Jul
PB 173950	A440 Steel Joints Connected by A490 Bolts, August 1965. (Pa.).
PB 173951	A Dynamic Stress Study of the Aluminum Bridge Over the Appomattox River at Petersburg, October 1965. (Va.).
PB 173952	Behavior of Large Bolted Joints, August 1965. (Pa.).
PB 173958	Traffic-Linkage Patterns Between a Metropolitan Area and the Communities Within its Region of Influence.
PB 173959	Response of Simple Span Highway Bridges to Moving Vehicles, September 1963. (Ill.)
PB 173960	Response of Three-Span Continuous Highway Bridges to Moving Vehicles, January 1964. (Ill.).
PB 173961	Computer Program Development, December 1965. (Tex.).
PB 173962	The Impact of Traffic on Residential Property Values and Retail Sales in Champaign-Urbana, Illinois, February 1965.
PB 173963	Sixth Street Freeway Traffic Study (1966). (Kansas City, Mo.).
PB 173964	Simulation of Traffic Flow as a Basis for Interchange Design.
PB 173965	Offtracking Calculation Charts for Trailer Combinations, January 1966.
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PB 174017	Vol. II
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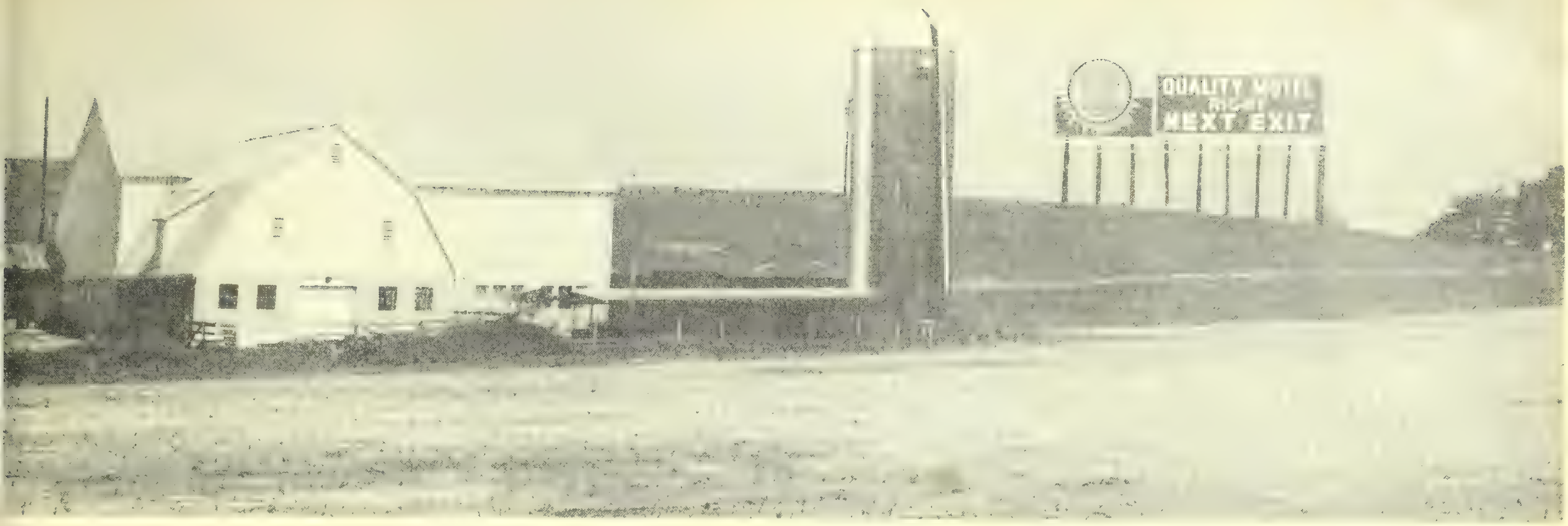
PB 174129	The Protected Paste Volume Concept Using New Air Void Measurement and Distribution Techniques.
PB 174130	Evaluation of Nuclear Moisture and Density Gages, June 30, 1966.
PB 174131	Exploration of Methods to Determine the Optimum Use of Indigenous Materials for Highway Construction, June 1966.

(*Continued on p. 238*)



On the rural landscape, giant billboards may rival large farm structures.

Billboards and Motorists' Needs

BY THE FEDERAL HIGHWAY ADMINISTRATION

Reported by[1] FLOYD I. THIEL, Economist
Office of Policy Planning

Introduction

ROADSIDE conditions along the highways today not only are complex, but some aspects of the involved roadside environment—continual littering, encroachment of vehicles on medians and other landscaped areas, proliferation of billboards in some areas, and appearance of jumbo billboards four times larger than their predecessors—seem discouraging. But amid the seemingly hopeless marring and cluttering of the landscape, there are a few hopeful signs, particularly concerning billboards. There seem to be fewer billboards, at least along some highways, especially those billboards that do not serve motorists' needs.

In the past, more attention seems to have been given to the number of billboards than to the type of information on the signs. For example, billboard regulation is much more concerned with the number of billboards permitted on the roadside than with the content of the sign messages themselves. Yet some signs deserve the attention of motorists more than others. Both critics and supporters of highway-billboard advertising can agree that billboards with gas, food, and lodging messages are more likely to serve motorists' needs than those that contain other messages. Also, it is fairly obvious that some billboards, regardless of regulations, will remain available for advertising in the future. What is not at all clear is the extent to which the available billboard space will be used to serve motorists' needs. Fragmentary information, however, indicates that billboard space, especially on the giant-size billboards that are springing up, is being used increasingly for highway-oriented messages rather than for general product advertising.

Some Disadvantages of Giant Billboards

Even though giant billboards now seem to be used primarily for highway-oriented advertising and may be more acceptable to motorists than if they were used for advertising that does not serve motorist needs, general use of these giant signs would be disadvantageous. Many of the esthetic objections to conventional billboards would be magnified if larger billboards were used. Furthermore, the proliferation of giant billboards tends to stymie other ways of communicating with motorists, like information sites, that would be less harmful to the landscape. A definite disadvantage would be the increased difficulty that small groups, like civic or service clubs and chambers of commerce, with small advertising budgets, would have in getting their messages to motorists because of the more expensive space on giant-size billboards. According to a study on motorists' information needs (*1*),[2] the cost of renting space on a giant-size billboard is $230 to $350 per month.

Changes in Highway Advertising

Highway billboard advertising is undoubtedly influenced by several factors, including billboard regulation. Some obvious changes in billboard-advertising practices usually can be traced to their causes. Others are harder to perceive or interpret, and their causes cannot be traced with any confidence, largely because the sample of information available for analysis is a small part of all the relevant information. The magnitude of the total information available is illustrated by the fact that about 1.1 million advertising signs have been erected along nearly 300,000 miles of Interstate and other primary highways by nearly 4,000 establishments that use outdoor advertising. The nature and size of the outdoor advertising industry is described in a Public Roads staff report issued in 1967 (*2*).

The large billboards, which are about 20 by 40 feet or more, giant size (12 by 24 feet regular size), and which are appearing some distance from roadsides in rural areas, are among the more recent developments in highway advertising. These signs seem to be beyond the 660-foot limit of the right-of-way referred to in the 1958 and 1965 legislation to control billboard advertising. They represent a disadvantage or a failure of

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[1] The work of several members of the Public Roads Office of Research and Development, especially John Yasnowsky and George Broderick, was used in this discussion.

[2] Italic numbers in parentheses identify the references listed on p. 229.

Giant billboards seem longer and higher than many rural buildings.

Table 1.—Billboards and highway mileage, 1966

	Illinois [1]		Iowa [1]		U.S. total [1]	
	Interstate	Other Federal-aid primary highways	Interstate	Other Federal-aid primary highways	Interstate	Other Federal-aid primary highways
Billboards [2] ...number	4,000	60,000	360	98,000	75,000	1,000,000
Mileage ...miles	890	11,000	380	10,000	21,000	223,000
Billboards per mile (both sides)..number	4	5+	1	10	3+	4

[1] Figures are rounded.

[2] *Number of Off-Premise Advertising Signs Along the Interstate and Federal-aid Primary Highways*, Bureau of Public Roads, Office of Planning.

The National System of Interstate and Defense Highways, improvement status of system mileage as of January 1, 1966, Bureau of Public Roads, Office of Engineering.

existing regulatory legislation. Regardless of what may happen to these giant billboards in the future, existing practice in States with billboard regulation differs noticeably from that in States without billboard regulation. Changes occurring in Texas, a State without billboard control, are discussed later.

Number of Billboards

The most obvious change in billboard activities is the decrease in the number of billboards along some types of highways. Along all Interstate highways inventoried in 1966, for example, billboards on both sides of the road averaged about three per mile. Along other Federal-aid primary (FAP) highways, they averaged about four per mile. (See table 1.) In selected States the difference in sign frequency along Interstate and other Federal-aid primary highways was very noticeable. In Iowa, there was an average of only one billboard per mile along Interstate highways, but nearly 10 per mile along other Federal-aid primary highways. (See table 1.) Of course, statistics on average numbers of signs per mile are misleading as signs often appear in clusters rather than at regular intervals.

An additional indication of changes occurring in highway-advertising practice was provided by a 1969 inventory taken at driving speed along a part of Interstate 80 in Iowa and Illinois. Along 200 miles of this road in Iowa there was an average of about one sign every 2 miles, nearly all giant billboards. (See table 2.) Between 1966 and 1969, the number of billboards along Interstate Highway 80 in Iowa declined from about one sign per mile to about one sign every 2 miles. (See table 3.) The State of Iowa has agreed to regulate billboard advertising under the Highway Beautification Act of 1965.

Although the number of billboards along Interstate Highway 80 in Iowa seems to have declined, in Illinois the number may have increased, but only slightly. (See table 3.) Along rural Interstate Highway 80 in Illinois, there is an average of nearly two billboards per mile, about 15 percent of which are giant size. The numbers of giant- and regular-size billboards on this road are shown in table 2. Illinois does not yet have an agreement to control billboards under the 1965 legislation, but it is one of the States that is qualifying for the bonus to control billboards under the 1958 legislation, which permits a number of exceptions for previously existing right-of-way, crossroad right-of-way, etc.

Analysis of recent experience in Texas provides some helpful insight into highway-advertising practices in a State that has no billboard control. Along nearly 1,000 miles of Texas rural Interstate highway, the number of signboard structures increased more than 17 percent from 1966 to 1968. There was no apparent change in the amount of space used to advertise motorist services, there were no giant-size billboards set back from the highway, but there was a substantial shift from the use of smaller signs to the use of larger signs.

Along Interstate routes on new locations in Texas, the number of billboards increased from 2.6 to 3.8 signs per mile, or nearly 50 percent, in the period 1966–68. Along old routes the increase was from about eight signs per mile in 1966 to nine per mile in 1968. Along 30 miles of Interstate Highway 45 between Houston and Galveston, a suburban area, the number of signs with their own supports increased 33 percent in the same 2- year period to more than 10 per mile. There were an additional five plus signs per mile without their own support—on fences, utility poles, etc., which "were alternately torn, tattered, rusty, and bent." (*3*).

Table 2.—Size of billboards along rural sections of Interstate 80 in Illinois and Iowa, 1969

	Illinois	Iowa
Billboards inventoried....number..	175	91
Regular-size billboards..........do....	148	3
Giant-size billboards............do....	27	88
Average billboards per mile ..do	2	[1] 0.5
Interstate mileage surveyed..miles..	95	203

[1] According to a 1967 inventory of the Des Moines-to-Davenport, 165-mile segment of Interstate 80, there was an average 0.6 signs per mile—85 giant-size signs, 25 standard-size signs, and three trailers used as signs. The inventory included both urban and rural portions of I-80 and was taken during a round-trip drive. The 1969 inventory reported here was taken during a one-way drive.

Table 3.—Number of billboards along rural areas of Interstate 80 in Illinois and Iowa, 1966 and 1969

	Inventoried in 1966 [1]		Inventoried in 1969 [2]	
	Total	Signs per mile [3]	Total	Signs per mile [3]
Illinois..................number	220	1.5	300	1.8
Iowa..................do	230	[4] 1	145	0.5

[1] 1966 sign inventory by State Highway Departments and the Bureau of Public Roads.

[2] Both sides of the right-of-way included in signs per mile. The 1969 figures are expanded from an unsystematic sample of approximately 60 percent of the I-80 mileage in Illinois and 66 percent in Iowa.

[3] Approximate mileage involved 150 in Illinois and 300 in Iowa.

[4] Based on estimated 250 miles opened to traffic.

Table 4.—Motorist exposure to advertising signs on Federal-aid primary highways, 1966 [1]

	Number of signs	Highway oriented advertising (gas, food, lodging)	Other advertising
Signboards inventoried—U.S., total..................number..	1,100,000		
Interstate:			
Signs [2]..................do....	75,000	[3] 40,000	35,000
Signs per mile..................do...	3+	2	1.5
Other Federal-aid primary highways:			
Signs [4]..................do..	1,000,000	[5] 250,000	750,000
Signs per mile..................do...	4	1	3

[1] 1966 inventory, State Highway Departments and Bureau of Public Roads. Signs inventoried included small signs.

[2] About 21,000 miles.

[3] Assuming that about 55 percent of the signs are highway oriented, as along Interstate 80 in Illinois in 1966.

[4] About 223,000 miles.

[5] Assuming that about 25 percent of the signs are highway oriented, as determined in a sample of 14 States.

Billboards Serving Motorists' Needs

There are indications that more billboard spaces are being used for motorists' needs than for general advertising, as suggested previously. According to logic, when fewer billboard spaces are available to communicate with passing motorists, more space will be used to convey gas-food-lodging messages that motorists may need, and less space to advertise tobacco, alcohol, fertilizer, insurance, and other products that motorists, as motorists, do not need.

Motorists' exposure to signboards, particularly the nonhighway or general types, seems to differ between highway systems and as time progresses. It was indicated by the 1966 sign inventory of all the States that nonhighway-oriented advertising appeared about one and a half times per mile of Interstate and about three times per mile for other Federal-aid primary highways (See table 4.) Of course many of the signs on Federal-aid primary highways were older and smaller than those on Interstate routes.

Along rural sections of Interstate Highway 80 in Illinois and Iowa, motorists are now apparently exposed to nonhighway-oriented advertising less frequently than they were before 1966. In 1966, a motorist could travel about 2.7 miles along I-80 in Iowa, and about 1.5 miles in Illinois, without seeing a nonhighway-oriented sign. In 1969, he can travel nearly 100 miles in Iowa and 5 plus miles in Illinois, without seeing such signs (table 5). Interstate routes are handling an increasing share of motor vehicle travel, which indicates that motorists' exposure to nonhighway-oriented billboards may be declining. Of course, Interstate Highway 80 may not be typical of all Interstate highways; its traffic seems to be increasing at a faster rate than is normal for most Interstate routes—35 percent and more compared with 6 to 8 percent for the U.S. overall. Motorists have better visibility on I-80 in Iowa and Illinois than on many other Interstate routes, but giant billboards beyond 660 feet are appearing even in areas like Maryland and Virginia where visibility sometimes is restricted by terrain or forests.

Motorists can make better economic and driving choices if they have enough useful information, but they cannot assimilate very many messages in a limited period of time. Consequently, some sign messages serve motorists' needs better than others, a concept that is supported by motorists' responses to queries about what types of messages they prefer or find acceptable. In one study, respondents were about twice as favorably

Strategically located billboards often appear in approaching motorist's direct view, although location is apparently 660 feet from the roadside.

Closeup view of giant billboard under construction shows relative size of normal-size man. According to the number of 4- by 8-foot plywood sheets and the size of supporting structure, the completed sign will be 84 feet long and 20–24 feet wide.

Table 5.—Motorist exposure to highway-oriented messages[1] and other messages along rural I-80 in Illinois and Iowa, 1966 and 1969

	Illinois		Iowa	
	Highway-oriented messages	Other messages	Highway-oriented messages	Other messages
	1966			
Signs inventoried ______ number	120	100	140	90
Length of highway inventoried ______ miles	150	150	250	250
Signs per mile ______ number	0.8	0.7	0.6	0.4
Distance between signs ______ miles	1.2	1.5	1.8	2.7
	1969			
Signs inventoried ______ number	157	18	89	2
Length of highway inventoried ______ miles	95	95	200	200
Signs per mile ______ number	1.6	0.2	0.5	0.01
Distance between signs ______ miles	0.6	5+	2.2	100

[1] Gas, food, and lodging primarily.

disposed toward highway service signs (51 percent) as they were toward signs advertising other products (27 percent) (4). In another study, 18 percent of the respondents regarded billboards in general as very useful, and 45 percent thought hotel and motel billboards were very useful (5). Respondents disapproved most billboard messages that do not serve motorists' needs.

In the past, approximately a quarter of all the advertising messages pertained to motorist services. According to the Public Roads Staff Report, p. 11 (3), in three separate studies by Texas A&M University, the University of Tennessee, and the Bureau of Public Roads, the percentages of billboards used for motorists' needs were determined to be 22, 33, and 26 respectively. Along rural Interstate Highway 80 in Illinois and Iowa, the 1966 inventory indicated that a higher percentage of billboards were used for highway-oriented messages—about 55 percent in Illinois and 60 percent in Iowa—and the 1969 survey along I-80 in Illinois and Iowa indicated that nearly all the classified billboard spaces were being used for motorists' needs. In fact, except for a Maytag message in Iowa, virtually 100 percent of the space on giant billboards is being used for highway-oriented messages like gas, food, and lodging. In another study in Iowa (1), it was also noted that most giant billboards were being used for motorist services. In 1966, a few signs that were larger than standard size had appeared in Iowa.

In table 6 the types of messages on billboards surveyed in 1969 along Interstate Highway 80 can be compared with messages on signs inventoried along I-80 and elsewhere in 1966. Highway oriented messages accounted for about 25 percent on all Federal-aid highways, for 55–60 percent on I-80 in 1966, and for 88–100 percent on I-80 in 1969. In table 7, sign messages along Interstate Highway 80 in Illinois and Iowa can be compared with those along all Federal-aid highways. As might be expected, a higher percentage of Interstate signs are being used for motorists' needs.

The increasing use of billboard space for motorists' needs is apparent when the 1966 and 1969 data for Interstate Highway 80 are compared. (See table 8.) The percentage of signs used for motorists' needs increased from 58 percent in 1966 to more than 90 percent in 1969, a difference that is unlikely to have occurred by chance—apparently less than one chance in five, as the difference tests significance at the twenty percent level. (See footnotes in table 8.) Indications that these apparent differences are a result of cause rather than chance obviously does not reveal what the cause is. A combination of causes is probably involved here—billboard regulation resulting in fewer and more expensive sites; insufficient time for signs to be erected on some new highways; industry's improved perception of what is needed, what is profitable, and what will be tolerated; and other changes in main-

Table 6.—Sign messages on certain Interstate and other Federal-aid primary highways

Survey or sample [1]	Signs used for motorist needs
1966 group	
	Percent
Federal aid primary highway in 14 States [2]	26
Texas Federal-aid primary highway [3]	22
Tennessee Federal-aid primary highway [4]	33
Interstate 80, rural:	
Illinois [5]	55
Iowa [5]	60
1969 group	
Illinois, I-80, rural signs:	
Regular size	88
Giant size	100
Iowa, I-80, rural signs:	
Regular size	100
Giant size	98

[1] Differences between the two groups are significant at the 5 percent level, using a Wilcoxon-Mann-Whitney test.

[2] Based on a Bureau of Public Roads sample, 1 percent of a 1966 highway sign inventory from 14 States Alabama, Alaska, California, Colorado, Connecticut, Indiana, Louisiana, Maryland, Michigan, Minnesota, Mississippi, Missouri, Montana, and Tennessee

[3] *Economic Effects of the Highway Beautification Program*, W. G. Adkins, W. F. McFarland, G. H. Meuth, J. T. Wynn, Texas A&M University, 1966.

[4] *The Economic Impact of the Highway Beautification Act on the Outdoor Advertising Industry, Landowners, and Selected Scenic Attractions in Tennessee*, L. S. Pipkin and F. L. Hendrix, University of Tennessee, 1966.

[5] All signs on I-80 shown on 1966 inventory of signs outside corporate boundaries.

[6] From 1969 sample of 60+ percent of I-80 mileage, taken at cruising speed.

tenance and in improved lighting that are occurring in the industry.

Conclusions

Although billboard practice appears to be changing, the apparent changes cannot be interpreted with any confidence because information is limited. Developments like giant billboards seem contrary to the intent of the highway beautification program. Efforts to ban billboards from the motorist's view rather than to restrict them within a limited distance of the highway, such as 660 feet, raise a question as to how long these billboards may exist. Even if they are permitted to remain on the landscape, current practice indicates that billboard regulation may cause most of the space to be used to serve motorists' needs.

Table 7.—Sign messages—Interstate and other Federal-aid primary highways

Survey or sample [1]	Signs used for motorist needs
Group 1—Federal-aid primary highways, including Interstate	
	Percent
All Federal-aid primary highways, 1966	26
Texas Federal-aid primary highways, 1966	22
Tennessee Federal-aid primary highways, 1966	33
Group 2—Interstate 80, Illinois and Iowa	
Illinois, I-80:	
Rural, regular-size signs, 1966	45
Rural, regular-size signs, 1969	80
Rural, giant-size signs, 1969	100
Iowa, I-80:	
Rural, regular-size signs, 1966	55
Rural, regular-size signs, 1969	100
Rural, giant-size signs, 1969	98

[1] Differences between the two groups are significant at the 5 percent level. For sources, see table 6

Whether a large proportion of the giant billboards would continue to be used for motorist needs, if these giant billboards are permitted to proliferate, is questionable; however, attention needs to be given now to the increasing number of them and, perhaps, to assuring that messages serving motorists' needs are given perference at locations where billboard space may remain available in the future.

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(3) *Texas Studies Relating to the Highway Beautification Program*, by W. G. Adkins et al., Texas Transportation Institute, Texas A & M University, 1968.

(4) *Estimates of the Impact of Sign and Billboard Removal Under the Highway Beautification Act of 1965*, by James B. Cloonan et al., University of Missouri, 1966.

(5) *Traveler Attitudes Toward Highway Billboard Advertising, A Survey of Selected Wyoming Motel Patrons*, by Dwight M. Blood, University of Wyoming, 1969.

Table 8.—Sign messages along rural sections of Interstate 80 in Illinois and Iowa, 1966 and 1969

Survey or sample [1]	Signs used for motorist needs
1966 group [2]	
	Percent
Illinois, all signs	55
Iowa, all signs	60
1969 group [3]	
Illinois:	
Regular size signs	88
Giant size signs	100
Iowa:	
Regular size signs	100
Giant size signs	98

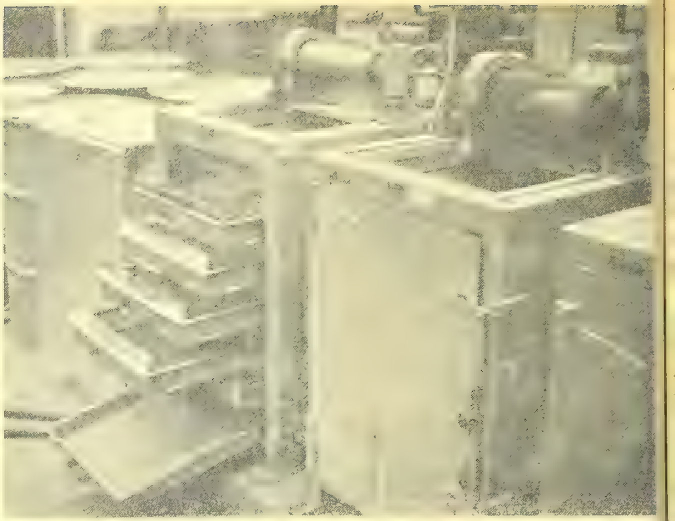
[1] Differences between the two groups are significant at the 20 percent level.

[2] All signs on I-80 shown on 1966 inventory of signs outside corporate boundaries.

[3] From 1969 sample of 60+ percent of I-80 mileage, taken at cruising speed.

Quality Assurance in Highway Construction

Part 5—
Summary of Research for Quality Assurance of Aggregate



Determining aggregate gradation by screen shaker (above) and sampling compacted aggregate base course (below)—two of the processes used to determine aggregate characteristics.

Reported by JAMES A. KELLEY,
Highway Research Engineer,
Materials Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

This is the fifth part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, Part 2.—Quality Assurance of Embankments and Base Courses, Part 3.—Quality Assurance of Portland Cement Concrete, and Part 4.—Variations of Bituminous Construction were presented in previous issues of PUBLIC ROADS. The remaining part, to be presented in the next issue, is Part 6.—Control Charts.

Introduction

A REVIEW of the evaluation by statistical techniques of highway aggregate characteristics is presented here as a condensed compilation of both historical data and data from designed quality-measurement projects in which the degree of conformance to specifications was statistically estimated. The historical data are not sufficient to determine the reason for any nonconformance to the specifications. However, the designed quality-measurement projects do provide data to determine quality at any point in a process, to disclose operations needing corrective action, and to give a valid estimate of specification conformance.

Reports from nine States on projects in which research data have been obtained are abstracted and summarized in this compilation to illustrate trends in gradation analysis, sampling and testing procedures, sand equivalent analysis as an alternate to gradation analysis, and soundness tests for aggregate quality.

Aggregate Base Course Characteristics

Specifications for base course aggregate usually contain limits for gradation, plasticity, soundness, and amount of deleterious material. Variations in gradation have been studied rather extensively to ascertain the degree of conformance obtained in construction. The data have been analyzed statistically to determine the variation in the material itself and that arising from sampling and testing. Most of the studies have been projects sponsored cooperatively by Public Roads and State Highway Departments, although some have been entirely State funded.

Nonuniformity of the final product has been disclosed by results of studies of gradation of different aggregate types including gravel, sand-gravel, and crushed stone. Differences in gradation were found between samples taken from the borrow pit or quarry plants, from the material after stockpiling, and again, from the material after it had been processed and compacted in place on the roadway. Differences in test results on the aggregate often resulted from the sampling method — sampling from a moving or stopped belt compared with sampling from a loaded truck. Representative sampling from an operation or placement also gave results that differed from those obtained by random sampling.

Combined variations frequently add up to a total variance of such magnitude that assurance of compliance with specifications is doubtful. However, with the knowledge provided by statistical analysis, it has been possible not only to pinpoint areas or operations requiring improvements, but also to determine when to take immediate corrective measures to assure better compliance.

Variance in historical data

Early statistical studies were made on data in office files of completed projects. Although this type of data was not randomly selected, statistical analysis usually disclosed that measurements of base course characteristics followed a normal distribution.

In table 1, which was extracted from a study of historical data for 257 observations of type A base in Louisiana, it is shown that for projects considered acceptable, the mean of the distribution for all sieve sizes was well within design limits. However, the statistically computed percentage of material within the design limits varied for each sieve size. The lowest value was 82 percent for material passing the No. 40 sieve. The highest value was 99 percent for material passing the ¾-inch sieve.

Variance of controlled research data

In the State of West Virginia, new construction was evaluated statistically to determine variations from design gradations. Analysis of variance disclosed that the material variance tended to be large and the sampling and testing variances small. According to the data, the magnitude of variance seemed to be directly related to the amount on each sieve.

The data in table 2 are an example of many studies in West Virginia and other States in which the components of variance are isolated by statistical analysis of field data on aggregate gradation characteristics. In figure 1, which is a diagram from the West Virginia report, proposed 95 percent tolerance limits are shown. The tolerances are ±13 percent on the sieve having approximately 50 percent of the material passing, and taper in both directions toward 0 percent and 100 percent passing where the tolerances are ± 2 percent.

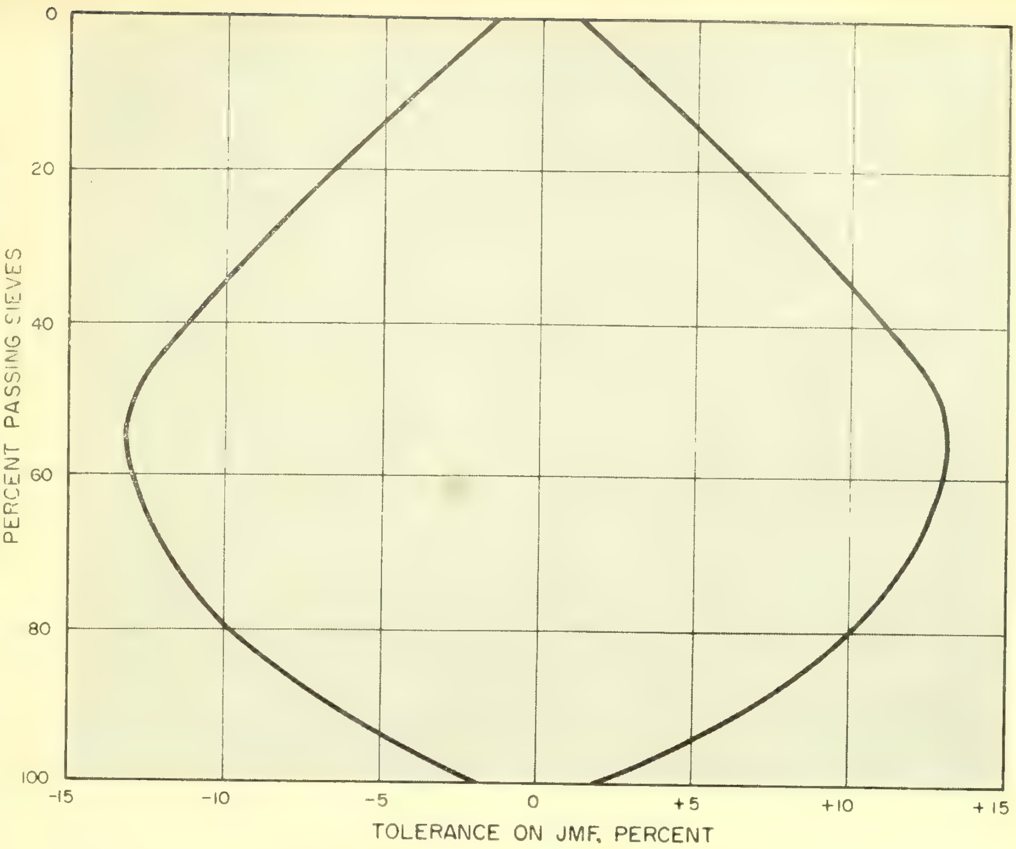


Figure 1.—Aggregate base course gradation characteristics, 95-percent probability tolerances on job-mix formula, West Virginia, 1966.

Variance caused by operators, sampling methods and equipment

Variance in the gradation of aggregate mixtures often is the result of sampling and testing procedures, as well as of the material itself. Several States have made quantitative measurements of these parameters. In Michigan (*1*) [1] a field experiment was carried out to determine what part aggregate inspectors, screening sieves, and sampling methods play in the uniformity of gradation results. A mathematical model was prepared to analyze the variations and ascertain whether (1) inspectors require further training to sample and test aggregates, (2) testing equipment requires periodic calibration or maintenance, (3) improved precision is feasible in gradation analysis, and (4) significant interactions occur in the experimental work. The results of this study were as follows:

- Individual inspectors and methods of sampling had a relatively small effect on gradation results on the ⅜-inch sieve. According to an analysis of components of variance, an estimated 4 percent of the total variance was attributable to inspectors, 6 percent to sampling methods, and the remaining 90 percent to inherent material and experimental deviations.
- For material passing the No. 10 sieve, significant interaction effects among the main factors of the experiment were shown to exist. Variance of 0–8 percent was due to methods of selecting samples, variance of 7–18 percent was due to testing and the remaining variance was attributable to inherent material and experimental deviations.
- The results of the analysis of variance (see table 3) indicated that interaction effect was significant enough to reduce the accuracy of major comparisons. According to the data in table 3, the combined influence (interaction) of inspectors and screening kits affected the gradation results. Also, the State found that the difference between the two sampling methods was large enough to be of

Table 1.—Base course analysis, gradation type A—historical data, Louisiana

Sieve size	Design limits	Mean distribution (X̄)	Standard deviation (σ)	Compliance with design
	Pct.	*Pct.*	*Pct.*	*Pct.*
¾ in.	75–95	90	2.5	99
No. 4	40–60	55	4.9	91
No. 40	20–45	37	6.3	82
No. 200	10–20	16	2.9	91

Table 2.—Base course gradation analysis—research data (n = 136), West Virginia

Sieve size	Design limits	Mean distribution (X)	Standard deviation (σ)	Variance		
				Material (σ_a^2)	Sampling (σ_s^2)	Testing (σ_t^2)
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1½ in.	100	100	0.0	0.0	0.0	0.0
¾ in.	40–85	80	3.9	9.6	5.8	0.2
⅜ in.		50	5.2	18.3	6.4	2.4
No. 4	20–60	34	4.3	12.7	4.9	1.1
No. 16		20	3.6	9.1	3.1	0.9
No. 40	5–25	11	2.8	5.7	2.6	0.0
No. 100		6	2.7	4.8	2.9	0.0

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[1] Italic numbers in parentheses identify the references listed on p. 237.

Table 3.—Analysis of variance for passing No. 10 sieve, Michigan [1]

Nature of effect	Source of variance	Sum of squares	Degrees of freedom	Variance estimate	F	F tests F 0.05	F tests F 0.01
Main factors	M	97.98	1	97.98	10.67	3.90	6.81
	I	39.10	2	19.55	2.12	3.06	4.75
	S	14.06	2	7.03	0.77	3.06	4.75
Interactions among factors	MI	25.29	2	12.64	1.38	3.06	4.75
	MS	3.61	2	1.80	0.20	3.06	4.75
	IS	280.20	4	70.05	7.63	2.43	3.45
	MIS	98.81	4	24.70	2.69	2.43	3.45
Replication	Residual	1,487.32	162	9.18			
	Total	2,046.36	179	11.43			

[1] Michigan Report No. R 571.
[2] M Sampling methods.
[3] Significant at the 1 and 5 percent levels (highly significant).
[4] I Aggregate inspectors.
[5] S Screening kits.
[6] Significant at the 5-percent level.

practical importance. The relative performance of aggregate inspectors was not consistent for all screening kits. These variances were significant, although not as large as the material variance, and it was presumed that, with training and corrective maintenance, the amount of testing and sampling variance could be reduced.

Methods of automatic aggregate sampling from a belt delivery system, and the variance resulting from the method used to prepare the test sample were studied in Idaho. Samples obtained with an automatic sampling device produced lower variance than those obtained manually, and the variance was more uniform. A direct relation was found between the splitting method and the testing variance of samples. Cross-split samples had a lower variance than those split only once. Cross-splitting is similar to quartering on a mat and combining the opposite quarters to form a single sample. Researchers tested 34 samples from Pit Le–111, collected by the manual method, and 25 samples from Pit Jr–2, obtained with an automatic sampling device. The variances for Jr–2 are relatively small and much more uniform than those for Pit Le–111. Part of the difference was attributed to the difference in the splitting techniques. The Idaho report was prepared to permit several cross comparisons of testing and sampling work. On the basis of these tests, 17 percent of the overall variance was due to testing variance whereas 30 and 53 percent, respectively, were due to sampling and material variances.

In Idaho, extensive research (*2*) was also conducted to ascertain whether the sand-equivalent test procedure was sufficiently reproducible to determine aggregate acceptability. The tests performed on cross-split samples at the Moscow laboratory resulted in a testing variance of 0.96, whereas the single-split samples at the Boise laboratory resulted in a testing variance of 1.85. For sand-equivalent determinations, considerable discrepancy existed between the results of the two laboratories; however, the test was considered satisfactory if the cross-split technique of the Moscow laboratory was used. As a result of the statistical analysis, improvements in [illegible] sampling and test methods were initiated.

A study in California (*3*), was undertaken to evaluate the effectiveness and reliability of the sand-equivalent tests used for procedure control and for measuring the variation of the aggregate investigated. Tests were performed on 200 random samples from each of six projects. Gradation was determined for each sample, and the analysis of variance was reported for the results on several sieves. It was concluded that the sand-equivalent and sieve analyses, supplemented by the R-value results in borderline situations, can provide satisfactory control of base and subbase material. The variances for the test results on the base material were generally smaller than the variance for the subbase material, perhaps because of the greater selectivity used for base material. Although the sampling and testing variances were relatively small for both materials, the testing variance was significantly larger than the sampling variance.

The results of this research were used to propose revision of California aggregate specifications. The proposed revisions, shown in table 4, were designed so that present specification limits could be retained by basing acceptance on a moving average of the five most recent test results. Broader limits for individual test results were established. Based on information available to him, the resident engineer is now authorized to accept the material, provided that the average indicates that the process is in control, even though a single test result may deviate from the broader limits.

According to the California report, class 2 aggregate base had an average sand-equivalent value of 44 with a pooled standard deviation of 4.8, and class 2 aggregate subbase had an average sand-equivalent value of 32 with a pooled standard deviation of 5.0. The proposed specification requirements for the sand-equivalent test and gradation are shown in table 4. It was stated in the report that:

"... the proposed specifications are to be used as guidelines only and are not intended to interfere with the present practice of designing specifications to meet local conditions for economic reasons. Once the gradation limits are established for a particular job, statistical specifications can be designed using the

Table 4.—Digest of proposed specifications for class 2 base and subbase aggregates, California

Material	Sand-equivalent values (Test Method, California 217) Minimum average [1]	Not to be lower than [2]	Overall average [3]	Gradation values Sieve size	Percent passing Moving average	Individual test result
Base	30	25	36	1 inch	..	100
				¾ inch	95±5	95(+5)(−7)
				No. 4	45±10	45±15
				No. 30	20±10	20±13
				No. 200	5.5±3.5	5.5±4.5
Subbase	23	18	30	3 inch	...	100
				2½ inch	95±5	95(+5)(−10)
				No. 4	65±25	65(±35)
				No. 200	12.5±12.5	12.5(+17.5) (−12.5)

[1] Five consecutive tests, each performed on independent sample.
[2] No single sand equivalent result to be lower.
[3] Overall average should be maintained for 99.9 percent probability of acceptance of suitable material.

Table 5.—Summary statistics for magnesium sulfate soundness tests, New York

Sand No.	Arithmetic means [1] Drying period 6-hour	30-hour	Difference in variation [2]	Higher	Variance estimates [1] Drying period 6-hour	30-hour	Difference in variation [2]	Higher
	Pct.	*Pct.*			*Pct.*	*Pct.*		
1	5.64	5.90	Insignificant	....	0.23	0.24	Insignificant	
2	17.09	15.74	do	....	0.05	1.83	Significant	30-hour.
3	23.21	23.83	Significant	30-hour	0.06	0.24	Insignificant	
4	17.01	41.98	do	6-hour	1.37	2.88	do	
5	47.65	50.36	do	30-hour	2.13	0.40	Significant	6-hour.

[1] Calculated from results of tests on two groups of three samples each.
[2] Statistical significance at 0.05 confidence level.

Table 6.—Results for surface mixture samples, South Carolina

Sieve size	Sample location	Specification limits percent passing	Control chart values		Standard deviation		Analysis of variance			
			Average ($\bar{X}$)	Total (σ)	Average ($\bar{x}$)	Total (σ)	Total (σ_T^2)	Material (σ_m^2)	Sampling (σ_s^2)	Testing (σ_t^2)
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
½ in.	Plant[1]	87–97	92.0	2.88	91.8	3.10	9.62	2.45	0.0	7.60
	Spreader[1]		90.0	4.00	90.7	3.52	12.41	8.74	0.0	3.50
	Compacted[1]		92.2	3.30	92.1	3.16	10.02	0.0	2.18	6.19
No. 4.	Plant	58-72	66.1	3.90	66.8	3.92	15.37	9.70	0.0	5.15
	Spreader		65.2	5.71	65.7	5.84	34.06	30.9	0.0	4.75
	Compacted		65.0	4.32	65.2	4.28	18.34	0.0	10.18	8.11
No. 10	Plant	42–58	52.0	3.45	52.6	3.69	13.59	8.33	1.76	3.69
	Spreader		53.0	4.98	52.7	5.60	31.35	28.15	0.0	5.04
	Compacted		54.3	3.92	54.5	4.01	16.11	0.0	11.36	7.39
No. 40.	Plant	21–35	28.1	1.70	28.3	1.91	3.64	2.28	0.0	1.40
	Spreader		28.0	1.41	28.5	2.34	5.48	4.92	0.0	1.00
	Compacted		28.7	1.85	28.8	2.08	4.31	0.0	2.51	2.09
No. 200	Plant	4–10	5.7	1.06	5.8	1.15	1.32	1.15	0.0	0.18
	Spreader		5.7	1.20	6.34	1.38	1.91	0.0	0.0	1.44
	Compacted		6.5	1.04	6.4	1.03	1.05	0.30	0.48	0.20

[1] Number of tests performed: Plant = 40, Spreader = 24, Compacted = 128

standard deviation as reported in this study, if no more accurate measurements are available."

After publication of the report, the State Division of Highways used similar specifications in its construction of projects.

Salt soundness test of aggregate

In certain uses, the quality of individual aggregate particles is an important characteristic, and owing to the composition of gravel or stone, the soundness of the aggregate pieces must be determined by certain standardized tests. In a study of *Salt Soundness Tests for Fine Aggregate* (*4*), the New York Department of Transportation used statistical concepts to investigate the procedures for determining both the sodium and magnesium salt soundness of fine aggregate and the methods used to judge the acceptability of a source. Data were presented on (1) the effect of drying time on the magnitude and reproducibility of test results, (2) overall reproducibility of the test with sodium and magnesium sulphates, and (3) the combined effect of testing and production variations on the scatter of test results from single sources. The summary statistics for the soundness tests, with various drying periods, is shown in table 5.

The conclusions extracted from the New York study were "(1) that an increase in drying time in the test from 6 to 30 hours will result in no change in the magnitude or reproducibility of the test results, (2) the reproducibility of the test with sodium sulphate and the test with magnesium sulfate are not significantly different, and (3) that it is possible to place the acceptance of sources of fine aggregate on a sound statistical foundation." They recommended that "the magnesium sulphate soundness test continue to be performed at the rate of one cycle per day and that the test with sodium sulphate be discontinued."

Even though the results of the New York study of fine aggregates were generally acceptable as reproducible results, many States have not obtained satisfactory correlation between salt-soundness test results and performance. This is particularly true for coarse aggregates.

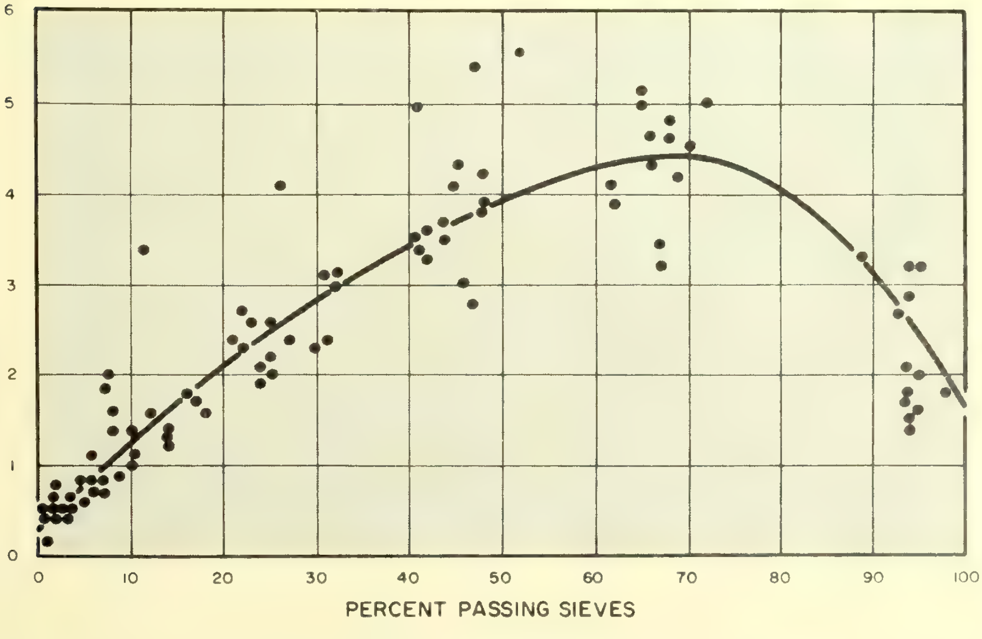

Figure 2.—Relation of standard deviation to percentages passing sieves, asphaltic concrete wearing course, West Virginia, 1966.

Bituminous Concrete Aggregate Characteristics

Aggregate used in bituminous concrete mixtures is subjected to several manipulations and treatments that are not applied to base course aggregate. The aggregate is heated for drying and mixing with asphalt. Often, it is stockpiled or placed in storage bins before the mixing operations. The final mixture is spread by a mechanical spreader and then a high force is applied for final compaction and rolling. Thus, the finished layer has experienced many abrasive forces that could cause not only changes in gradation of the aggregate component, but also changes in density and stability. A more detailed analysis of variations in aggregates used in bituminous construction is contained in Part 4. However, the more important findings of individual projects are reported here.

In a study performed in South Carolina (*5*), random samples of asphalt mixtures were selected from trucks at the batch plant, from the roadway just behind the spreader, and from the roadway after compaction, to determine whether any progressive change occurred in the characteristics of the aggregate. A summary of this work is given in tables 6 and 7 in which the specification limits and analysis of variance for both surface and binder courses are also shown. The aggregate passing the No. 4 sieve in the surfacing mixture was within the job-mix formula only 50 percent of the time by routine control sampling and 66 percent of the time by random sampling. The material passing the No. 40 sieve was within the job-mix formula 76 percent of the time by control sampling and 88 percent of the time by random sampling. The test results shown in table 6 indicate that the average for the No. 4 material varied from 66.8 to 65.2 percent whereas the No. 40 material varied only from 28.3 to 28.8 percent, conforming more closely to the job-mix formula. The greatest standard deviation occurred on the samples from the spreader box.

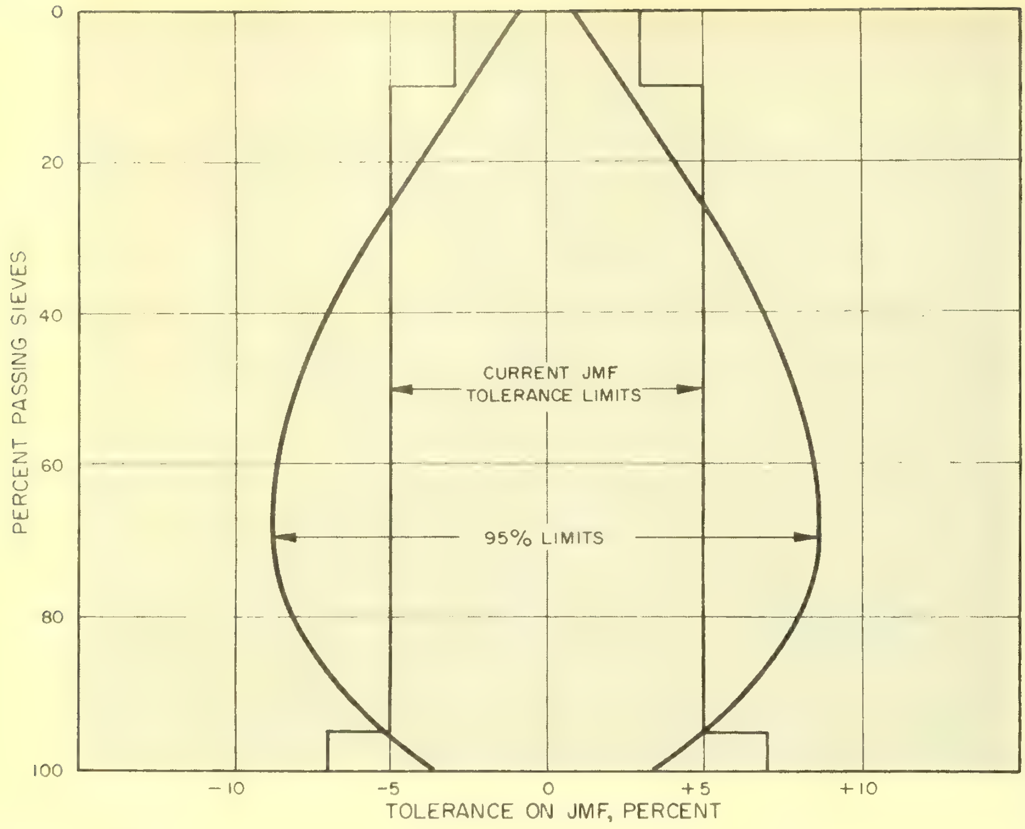

Figure 3.—Relation of 95-percent probability tolerances on job-mix formula to percent passing sieves, asphaltic wearing course, West Virginia, 1966.

The characteristics of aggregate used in bituminous mixtures were also explored in West Virginia (*6*). An analysis of the aggregate passing the No. 4 sieve is shown in table 8 for 10 bituminous projects. For the percent passing the No. 4 sieve, nine of the 10 projects had an average value that was within the specifications. However, the overall standard deviation for the individual projects was so large that many of the projects had a considerable amount of nonconforming material. Because of the large overall standard deviation, 4.4 percent, a change in the specified job-mix tolerances was recommended.

The following excerpts were taken from the West Virginia report:

"Tolerances for percentages passing other sieves may also require adjustment. Inspection of the data shows that the major component of the overall standard deviation, σ_o, is the materials variance, σ_a, and sampling and testing can be reduced to a negligible amount."

"The size of the standard deviation of the percent passing any sieve, neglecting sampling and testing error, depends to a large extent upon the value of the percentage passing that sieve" (fig. 2).

In the West Virginia report it was proposed that tolerances for gradation specifications be varied according to the percentage passing any sieve. The magnitude of variation to provide 95 percent probability tolerances on the job-mix formula is shown in figure 3.

Other States engaged in statistical studies of aggregate-gradation characteristics in bituminous mixtures have indicated that, for best uniformity and smallest standard deviations, control of gradation should be at the mixing plant. Job-mix tolerances for all gradations should be adjusted for the percentages expected to pass the specified sieves.

Portland Cement Concrete Aggregate Characteristics

Because structural concrete in highway construction is critical, the specified aggregate gradation should be assured. Several research projects were conducted to determine the best place to sample aggregate for control, permissible tolerances on various sieve sizes, and alternate methods or tests to establish gradation uniformity.

In California, a study was performed to determine the precision of current test methods and the feasibility of using statistical quality control procedures for portland cement concrete aggregate. Several conclusions were drawn from this study: Present controls and specifications for aggregate gradation need to be modified because of the high material variance and large percentages of out-of-specification gradation; sand-equivalent and cleanliness test methods were satisfactory; more efficient field control would be possible if control charts were used; better control of gradation could be obtained by using a *moving average* based on the results of the five most recent individual tests; material and testing variances were considerably larger than were anticipated (see fig. 4); and a relatively high percentage of the aggregate failed to meet the specification, which is shown by the diagram in figure 5.

A statistical analysis of variance in aggregate for portland cement concrete was made by Louisiana (*7*). The variations in gradation of fine and coarse aggregate sampled from different stockpiles as well as the differences be-

Table 7.—Results for binder mixture samples, South Carolina

Sieve size	Sample location	Specification limits percent passing	Control chart values		Standard deviation		Analysis of variance			
			Average ($\bar{x}$)	Total (σ)	Average ($\bar{x}$)	Total (σ)	Total (σ_{to}^2)	Material (σ_m^2)	Sampling (σ_s^2)	Testing (σ_t^2)
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1 in.	Plant[1]	80–97	93.4	3.95	93.9	4.13	17.09	0.0	2.60	13.04
	Spreader[1]		93.8	5.19	93.3	4.07	16.56	3.35	0.0	12.25
	Compacted[1]		93.4	4.70	93.7	4.43	20.07	4.59	0.0	15.43
No. 4	Plant	35–50	40.6	4.76	40.8	4.61	21.25	8.47	5.37	7.52
	Spreader		40.1	6.79	41.3	6.01	36.14	21.20	0.0	12.87
	Compacted		43.1	4.42	42.9	4.54	20.60	4.23	7.36	8.67
No. 16	Plant	25–35	32.2	4.06	32.2	3.90	15.24	7.22	3.68	4.43
	Spreader		32.2	5.46	32.9	4.83	23.3	14.03	0.0	8.38
	Compacted		34.8	3.51	34.6	3.74	13.96	0.0	5.86	5.91
No. 40	[illegible]	None	18.0	2.09	18.1	2.18	4.74	2.17	1.14	1.46
	[illegible]		18.3	2.66	18.5	2.34	5.48	3.18	0.0	2.37
	[illegible]		19.7	2.00	19.9	3.62	13.15	2.24	0.0	10.87
No. 200	[illegible]	None	4.1	0.53	4.1	0.54	0.30	0.17	0.05	0.09
	[illegible]		4.0	0.49	4.1	0.58	0.33	0.18	0.0	0.18
	[illegible]		4.3	0.67	4.3	0.67	0.44	0.11	0.18	0.15

[1] Number of tests performed [illegible] compacted – 380.

tween samples within stockpiles, were determined. According to the Louisiana report, "The largest component of variance is between stockpiles, which is reflective of material variance. The variation between samples within stockpiles can be attributed to either the stockpiling technique or sampling procedure." The actual results for the fine aggregate passing the No. 4 sieve are shown in table 9. The analysis of the coarse aggregate was similar to that of the fine aggregate. As shown in table 10, heavily loaded sieves had the greatest deviations and the largest amounts of material outside the specification limits. As a result of this study, the researchers prepared suggested acceptance limits and frequencies of measurement for aggregate used in portland cement. (See table 11.)

Quality control of aggregate used in portland cement concrete by sampling from the stockpiles and bins at the central plant was studied in Oklahoma (*8*). The dry aggregate was weighed at the bin site, the cement added, and the batch hauled by trucks to the road site where the concrete was mixed. Random samples were taken at a point in the stockpile nearest the bins. The analysis of the gradation indicated that the mean values for each sieve size were within the specification limits although many individual values were outside

Table 8.—Analysis of variance of bituminous concrete aggregate for 10 projects in West Virginia

Project No.	Sample location	Number of samples (n)	Percent passing No. 4 sieve Specification 60–70				
			Average ($\bar{X}$)	Overall standard deviation (σ_o)	Standard deviation		
					Material (σ_a)	Testing (σ_t)	Sampling (σ_s)
			Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*
38A1	Even	96	66.7	2.5	2.5	0.0	0.0
38A1	Odd	96	67.5	4.8	2.5	1.6	3.8
3235	Even	100	65.6	3.6	3.6	0.0	0.0
3235	Odd	100	64.8	3.3	3.3	0.0	0.0
3462	Truck	120	69.3	4.5	3.4	0.0	2.9
3462	Pavement	120	65.7	4.7	4.1	0.8	2.1
173 H(1) & (2)		200	70.0	4.8	3.6	1.5	2.2
204A & 204A(3)		180	72.1	5.1	4.4	1.8	1.7
284(C) & (4) SRC		120	61.7	4.0	3.9	0.8	0.3
284(C) & (4) AASHO		120	61.5	4.2	3.9	1.6	0.5
Average, all projects			66.5	4.4	3.7	1.2	1.9

Table 9.—Analysis of variance on gradation of fine aggregate for portland cement, Louisiana Percent passing No. 4 sieve

Source of variance	Sum of squares (SS)	Degrees of freedom (DF)	Mean squares (MS)	Estimate of mean squares[1] (EMS)	F.05
Between stockpiles	249.43	8	31.18	$\sigma_e^2+2\sigma_s^2+16\sigma_{st}^2$	[2]8, 63
Between samples within stockpiles	52.44	63	0.83	$\sigma_e^2+2\sigma_s^2$	[2]63, 72
Between subsamples within samples	14.35	72	0.20	σ_e^2	------
Total	316.22	143	------	----------------	------

[1] $\sigma_e^2=.20$(Testing) $\sigma^2_{sample}=.32$ (Sampling) $\sigma^2_{stockpile}=1.90$ (Material).
[2] Significant.

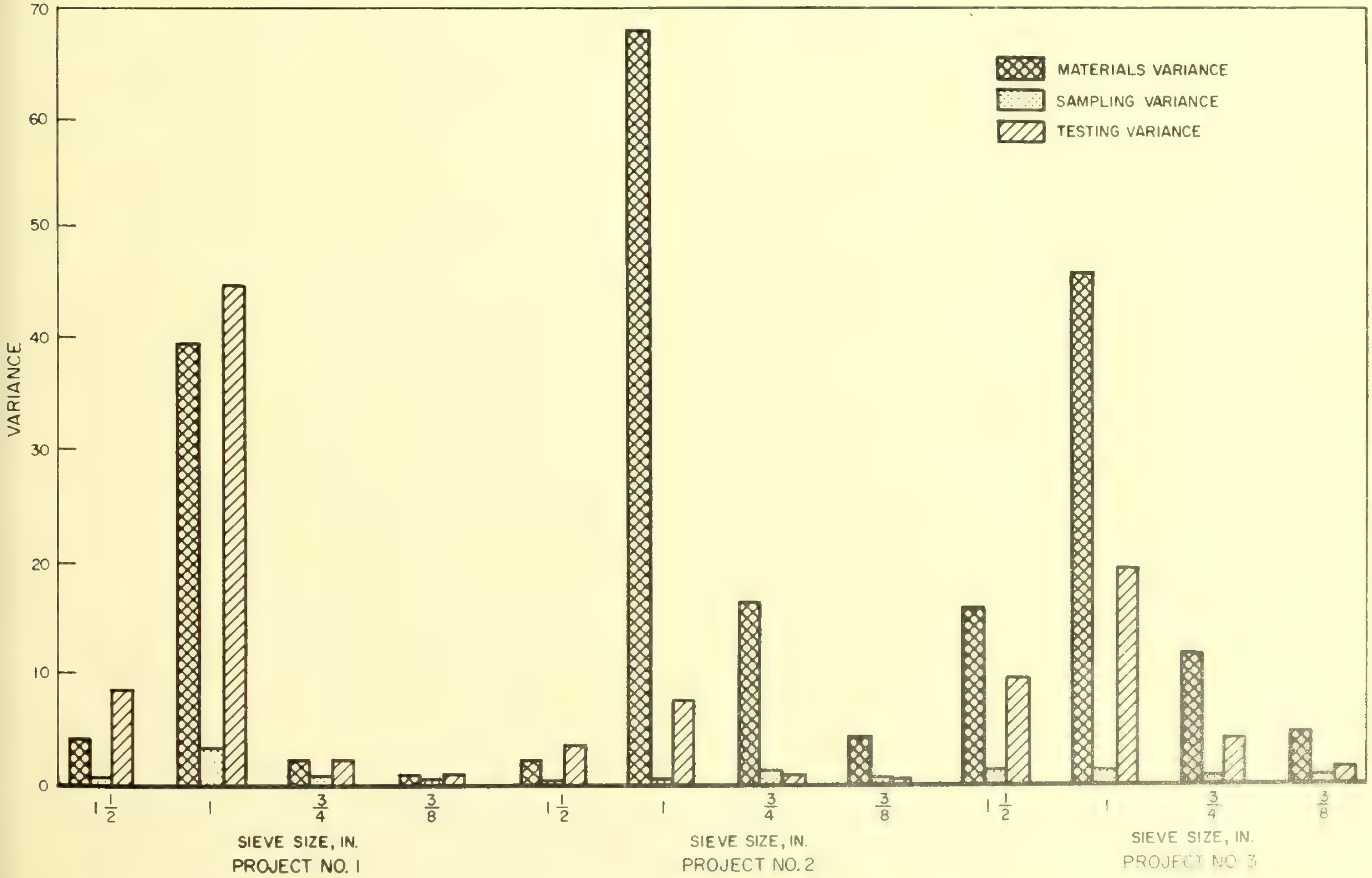

Figure 4.—Analysis of variance of portland-cement-concrete coarse aggregate for material passing different sieves, California.

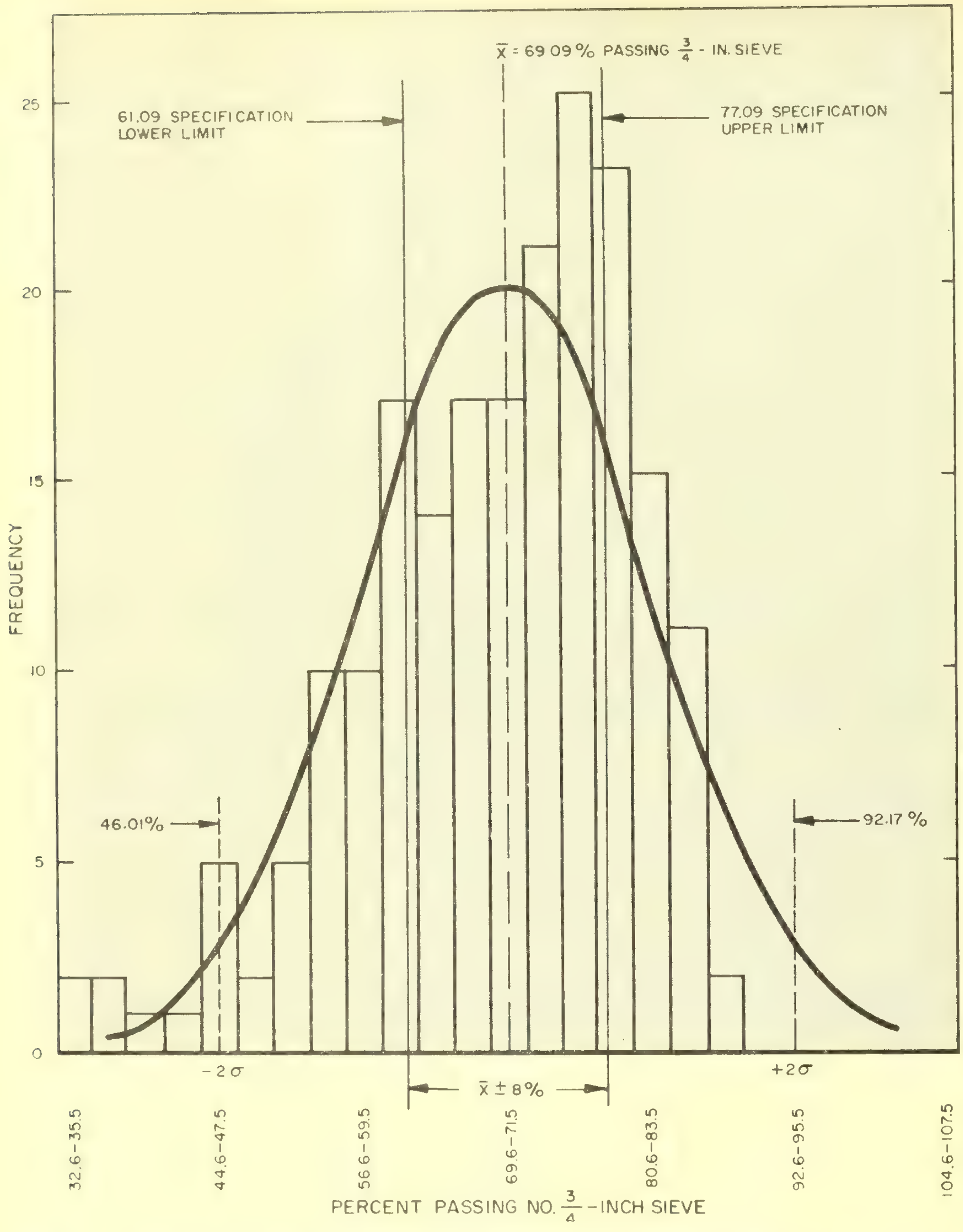

Figure 5.—Percent of material outside of specifications, portland-cement-concrete aggregate passing No. 3/4-in. sieve, project No. 2, California.

the upper and lower control limits. The Oklahoma Department of Highways recommended that the gradation determination be continued, but with certain modifications. Acknowledging that some plans provide for the sampling of aggregates at the batching bin, researchers pointed out in their report that sampling at the stockpiles permits early detection of undesirable or unacceptable aggregate, which is the purpose of quality control—to locate defective material as quickly as possible.

Summary

Some of the important findings from selected research on the characteristics of aggregate used in base courses and in bituminous and portland cement concrete mixtures have been presented here. More attention has been given to aggregate-gradation characteristics from source of supply to placement, than to other characteristics. Early studies concentrated on historical data; more recent studies were conducted during actual field construction. Comprehensive plans were devised to study historical data and to measure variability during construction. The degree of conformance to gradation specifications was found to vary from step to step in the processing. Analysis of variance usually was applied during construction to determine causes of the variation and to locate conditions needing corrective action.

Generally, the largest deviations from specifications were in the material in the middle of a stack of sieves—where a large amount of material is on individual sieves.

Knowledge of inherent material, sampling, and testing variations enables the engineer to design specifications with tolerances that are compatible with local conditions and, thereby, to avoid unenforceable requirements or unreasonable expense and still obtain a suitable aggregate.

For aggregate control, the sand-equivalent test rather than gradation is preferred in some States, and statistical research, conducted to ascertain whether the sand-equivalent test is informative and reproducible, has

Table 10.—Summary of statistical results on portland cement concrete aggregate gradation, Louisiana [1]

Sieve size	Average (x̄)	Standard deviation (σ)	Minimum	Maximum	Outside specifications	Specification limits	Variance (σ²)		
							Test	Sample	Stockpile
Grade A course aggregate									
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*		*Pct.*	*Pct.*	*Pct.*
[illegible]	95.6	3.8	82.7	99.9	7.3	90-100	0.84	5.65	9.97
[illegible]	75.4	10.8	46.1	88.8	2.1	40-88	6.41	77.62	40.72
[illegible]	35.5	12.7	4.6	60.2	13.5	15-55	9.18	132.04	26.72
No. 4	1.3	1.2	0.2	5.5	0.0	0-6	0.13	0.76	0.72
Fine aggregate									
No. 4	[illegible]	1.5	92.1	99.9	3.8	95-100	0.20	0.32	1.90
No. 16	79.2	7.9	56.6	91.6	9.7	45-90	0.70	10.72	57.69
No. [illegible]	[illegible]	6.5	7.2	31.6	1.4	7-30	5.36	39.88	0.0
No. 100	2.1	1.3	0.3	5.7	0.0	0-7	0.04	0.40	1.34

[1] Louisiana Department of [illegible]

Table 11.—Suggested acceptance limits for portland cement concrete aggregate[1]

Sieve size	Acceptance probability (P_a)	Rejection probability (P_r)	n	Acceptance limits				Measurement frequency
				Mean		Individual		
				LL	UL	X −	X +	
Gradation of fine aggregate, percent passing								
	Pct.	*Pct.*						
No. 4	99	90	4	95.00	99.75	3.93	3.93	One every 200 cu. yd.
No. 16	99	90	4	68.97	89.45	20.80	20.80	
No. 50	99	90	4	7.51	24.35	17.11	17.11	
No. 100	99	90	4	0.41	3.71	3.35	3.35	
Gradation of grade A coarse aggregate, percent passing								
1 in.	99	90	4	90.63	100.00	10.10	10.10	One every 500 cu. yd.
¾ in.	99	90	4	61.43	89.39	28.40	28.40	
½ in.	99	90	4	19.08	51.84	33.27	33.27	
No. 4	99	90	4	0.0	2.70	3.20	3.20	

[1] Louisiana Department of Highways Report, 1966.

confirmed its usefulness, although the amount of data at present is rather limited.

Statistical research on salt-soundness determination indicated that drying time did not need to be changed; that sodium sulfate testing could be discontinued, as magnesium sulfate testing is satisfactory; and that it is possible to place acceptance of fine aggregate sources on a sound statistical foundation.

In statistically-oriented research on aggregate gradation for bituminous mixtures, information similar to that for base courses was developed, and indicated that variation from the specifications differs according to the point of sampling. The research showed that the variation on certain sieve sizes is considerable, indicating either the need for improvement in sieving operations or the establishment of wide tolerances in the specifications to eliminate compliance disagreement. The research also indicated that sampling at the hot bins was preferred for process control, whereas, sampling at the compacted bituminous layer was best for establishing uniformity of the mixture.

Data in various studies indicated that the aggregate used in portland cement concrete had a smaller standard deviation than the aggregate used in bituminous mixtures or in base courses, but that statistical analysis provided information for early detection of undesirable gradation or undesirable quality.

Based on their studies, highway departments in some States are revising their specifications and outlining specific sampling procedures. The use of statistically designed control charts is highly recommended for control of the characteristics of aggregate by these States. The moving average of five most recent individual test results is reported to be the most practical for controlling the construction processes.

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(*8*) *Statistical Quality Control of Portland Cement Concrete Pavements,* prepared by Joakim G. Laguros, University of Oklahoma, for Oklahoma Department of Highways and U.S. Bureau of Public Roads, June 1968.

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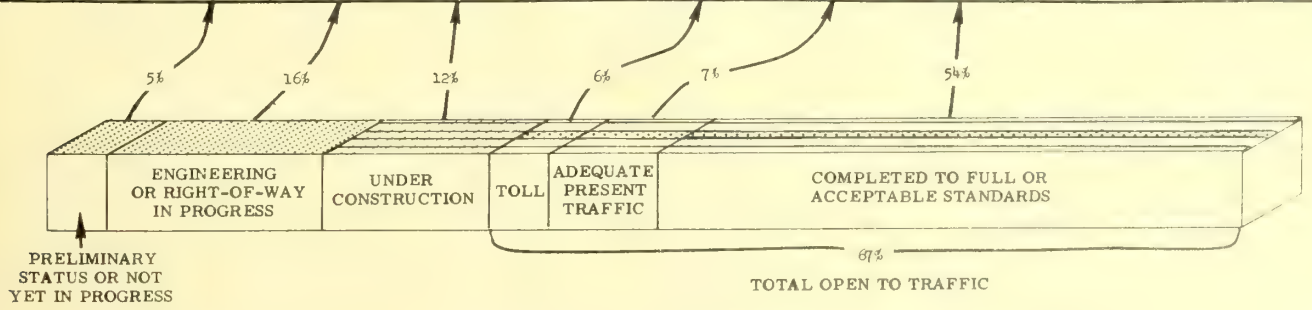
THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

IMPROVEMENT STATUS OF SYSTEM MILEAGE AS OF JUNE 30, 1969

TABLE I

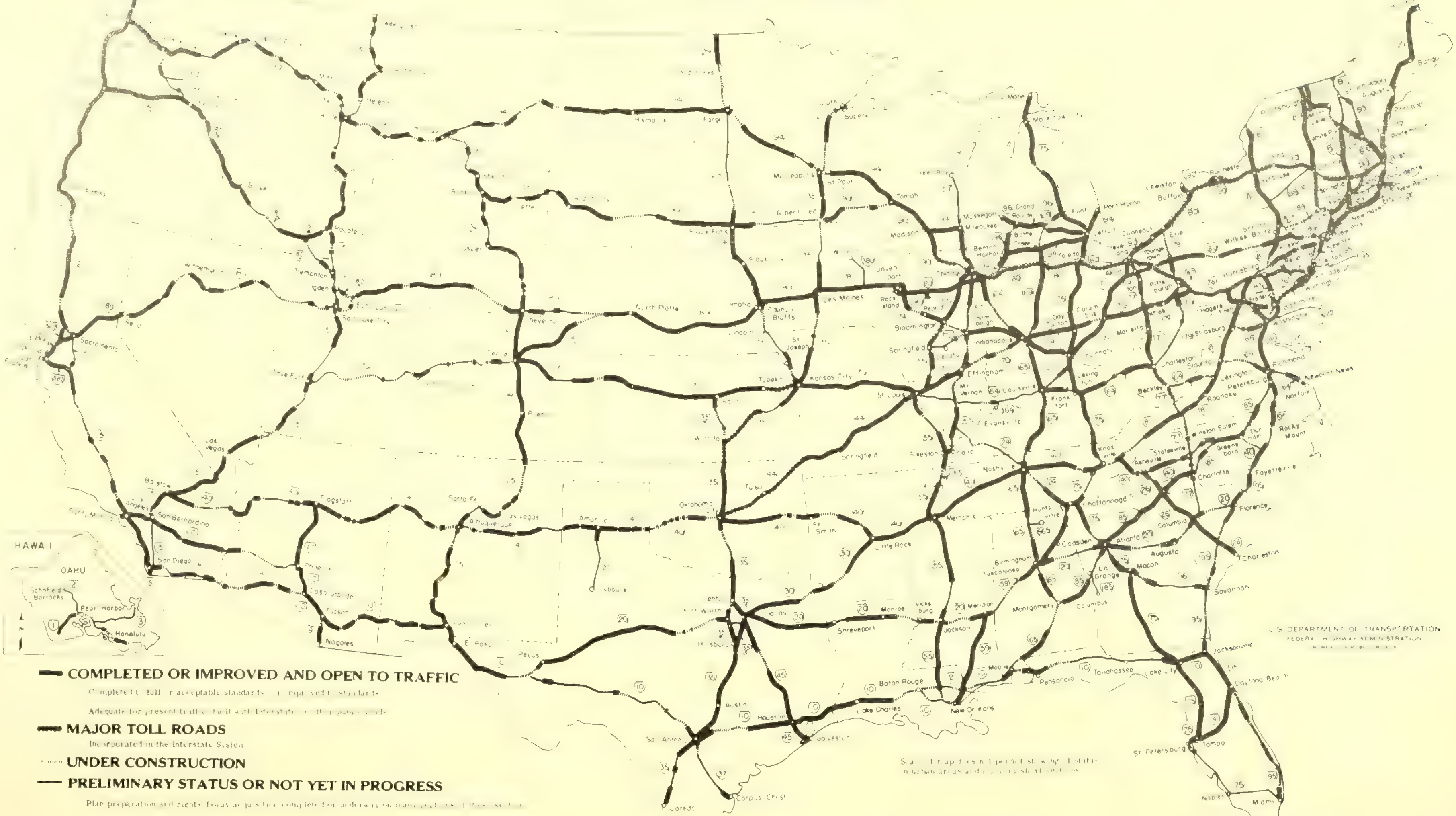
STATE	PRELIMINARY STATUS OR NOT YET IN PROGRESS 1/	WORK IN PROGRESS			OPEN TO TRAFFIC				TOTAL DESIGNATED SYSTEM MILEAGE	STATE
		ENGINEERING OR RIGHT-OF-WAY	UNDER CONSTRUCTION	TOTAL UNDERWAY	TOTAL FACILITIES	IMPROVED TO STANDARDS ADEQUATE FOR PRESENT TRAFFIC	COMPLETED TO FULL OR ACCEPTABLE STANDARDS	TOTAL OPEN TO TRAFFIC		
ALABAMA	19.20	192.11	191.20	383.31	-	143.90	350.20	494.10	896.61	ALABAMA
ARIZONA	5.90	157.83	201.30	359.13	-	226.04	581.10	807.14	1,172.17	ARIZONA
ARKANSAS	-	24.87	86.25	111.12	-	41.29	367.14	408.43	519.55	ARKANSAS
CALIFORNIA	103.80	334.70	338.50	673.20	10.20	281.90	1,205.00	1,497.10	2,274.10 2/	CALIFORNIA
COLORADO	143.86	111.00	66.66	177.66	-	112.24	542.12	654.36	975.88	COLORADO
CONNECTICUT	51.60	23.08	11.20	34.28	16.40	47.37	197.50	261.27	347.15	CONNECTICUT
DELAWARE	-	9.40	2.07	11.47	14.30	0.92	13.92	29.14	40.61	DELAWARE
FLORIDA	271.21	290.29	119.63	409.92	44.85	-	686.94	731.79	1,412.92	FLORIDA
GEORGIA	38.80	278.17	156.56	434.73	-	6.94	666.71	673.65	1,147.18	GEORGIA
HAWAII	11.60	21.68	3.46	25.14	-	1.57	13.54	15.11	51.85	HAWAII
IDAHO	-	120.60	93.88	214.48	-	96.30	300.78	397.08	611.56	IDAHO
ILLINOIS	83.82	264.57	272.53	537.10	155.66	148.05	798.63	1,102.34	1,723.26	ILLINOIS
INDIANA	14.00	164.12	175.34	339.46	156.90	15.39	603.37	775.66	1,129.12	INDIANA
IOWA	74.80	109.24	79.71	188.95	3.57	-	516.47	520.04	783.79	IOWA
KANSAS	19.60	80.50	70.10	150.60	185.90	0.30	464.10	650.30	820.50	KANSAS
KENTUCKY	-	145.42	111.46	256.88	39.20	3.40	439.12	481.72	738.60	KENTUCKY
LOUISIANA	30.00	168.67	178.44	347.11	-	6.35	319.76	326.11	703.22	LOUISIANA
MAINE	1.75	32.78	2.38	35.16	57.70	99.16	118.45	275.31	312.22	MAINE
MARYLAND	25.21	7.16	30.56	37.72	53.04	70.89	173.25	297.18	360.11 3/	MARYLAND
MASSACHUSETTS	19.07	29.43	33.01	62.44	134.41	27.36	223.70	385.47	466.98	MASSACHUSETTS
MICHIGAN	41.33	167.00	44.39	211.39	4.77	44.41	872.40	921.58	1,174.30	MICHIGAN
MINNESOTA	9.40	240.43	210.80	451.23	-	30.28	422.51	452.79	913.42	MINNESOTA
MISSISSIPPI	-	125.60	76.40	202.00	-	19.20	457.10	476.30	678.30	MISSISSIPPI
MISSOURI	26.60	242.30	43.40	285.70	0.30	160.80	672.50	833.60	1,145.90	MISSOURI
MONTANA	24.60	465.31	101.80	567.11	-	301.84	292.45	594.29	1,186.00	MONTANA
NEBRASKA	1.90	72.84	31.75	104.59	0.22	13.58	359.18	372.98	479.47	NEBRASKA
NEVADA	-	122.49	38.68	161.17	-	5.34	368.05	373.39	534.56	NEVADA
NEW HAMPSHIRE	11.30	25.32	6.06	31.38	22.02	14.76	135.63	172.41	215.09	NEW HAMPSHIRE
NEW JERSEY	46.00	90.60	62.40	153.00	46.30	26.40	113.50	186.20	385.20 4/	NEW JERSEY
NEW MEXICO	37.49	149.09	114.40	263.49	-	61.05	636.27	697.32	998.30	NEW MEXICO
NEW YORK	142.17	46.63	88.72	135.35	491.88	53.27	532.38	1,077.53	1,355.05	NEW YORK
NORTH CAROLINA	50.04	193.98	121.22	315.20	-	16.75	455.26	472.01	837.25	NORTH CAROLINA
NORTH DAKOTA	62.60	38.83	77.14	115.97	-	51.94	340.30	392.24	570.81	NORTH DAKOTA
OHIO	8.80	154.25	178.50	332.75	206.37	54.98	931.20	1,192.55	1,534.10	OHIO
OKLAHOMA	-	49.29	139.30	188.59	174.04	23.30	420.90	618.24	806.83	OKLAHOMA
OREGON	24.73	54.90	12.62	67.52	-	111.16	531.52	642.68	734.93	OREGON
PENNSYLVANIA	41.86	96.69	286.06	382.75	360.18	8.35	781.33	1,149.86	1,574.47	PENNSYLVANIA
RHODE ISLAND	26.50	7.63	15.43	23.06	-	13.81	36.82	50.63	100.19	RHODE ISLAND
SOUTH CAROLINA	73.70	92.13	133.72	225.85	-	15.14	441.14	456.28	755.83	SOUTH CAROLINA
SOUTH DAKOTA	-	161.39	93.20	254.59	-	60.28	364.36	424.64	679.23	SOUTH DAKOTA
TENNESSEE	7.50	254.65	116.55	371.20	-	119.40	547.00	666.40	1,045.10	TENNESSEE
TEXAS	122.71	529.53	375.06	904.59	-	272.14	1,866.78	2,138.92	3,166.22	TEXAS
UTAH	50.82	349.95	234.93	584.88	-	20.23	277.82	298.05	933.75	UTAH
VERMONT	-	96.32	51.06	147.38	-	4.43	168.57	173.00	320.38	VERMONT
VIRGINIA	9.80	214.19	153.26	367.45	37.60	44.87	608.48	690.95	1,068.20	VIRGINIA
WASHINGTON	68.81	118.23	24.74	142.97	-	178.99	363.39	542.38	754.16	WASHINGTON
WEST VIRGINIA	29.52	148.53	63.72	212.25	87.20	0.30	184.68	272.18	513.95	WEST VIRGINIA
WISCONSIN	105.47	1.73	39.14	40.87	-	24.71	392.10	416.81	563.15	WISCONSIN
WYOMING	73.83	72.73	97.87	170.60	-	30.31	638.90	669.21	913.64	WYOMING
DISTRICT OF COLUMBIA	9.91	7.57	2.04	9.61	-	2.92	7.15	10.07	29.59	DISTRICT OF COLUMBIA
PENDING	45.25 5/								45.25 5/	PENDING
TOTAL	2,066.86	6,955.75	5,258.60	12,214.35	2,303.01	3,114.31	22,801.47	28,218.79	42,500.00	TOTAL



1/ Public hearings have been held on route location, and location studies are underway on many portions of the mileage in this column.
2/ Excludes the 17.20 mile Century Freeway (I-105) which was added to the system under the "Howard Bill."
3/ Excludes 28.50 miles of the Baltimore-Washington Parkway (I-295) which was added to the system under the "Howard Bill."
4/ Excludes 27.40 miles chargeable to the "Howard Bill" of the total 34.40 mile Trenton-Asbury Park Spur (I-195) which was added to the system under that bill.
5/ Consists of mileage which has not been assigned to any specific route and is a reserve for final measurement of the system.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF JUNE 30, 1969



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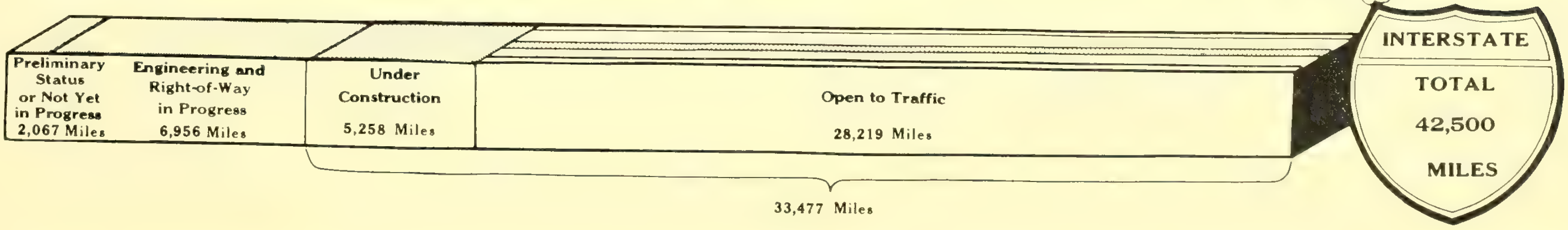

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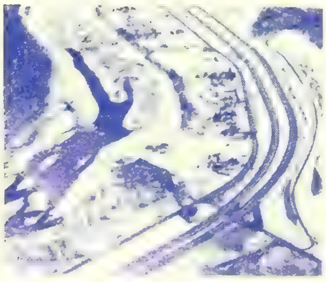

COVER

U.S. Highway 22-322 in Perry County, Pa., hugs the Juniata River (left) from Amity Hall to Lewistown. The controlled access highway replaces the old narrow road seen between the river and the new road. (Photo courtesy Pennsylvania Dept. of Highways)

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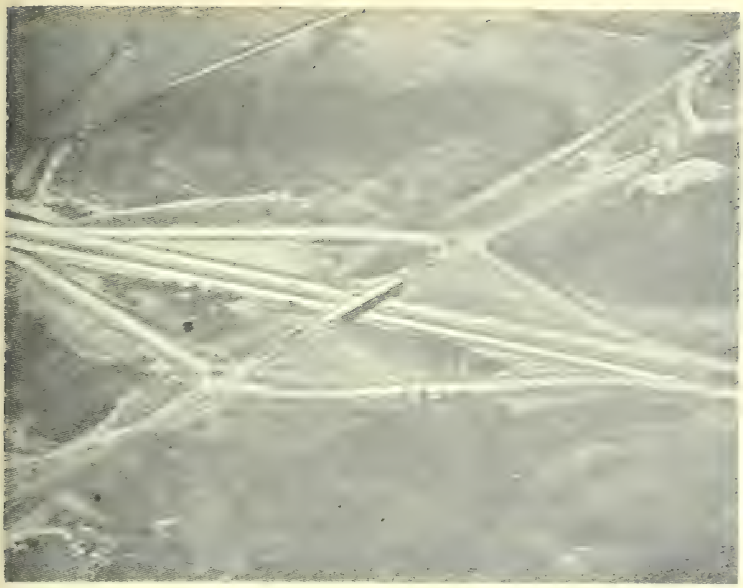

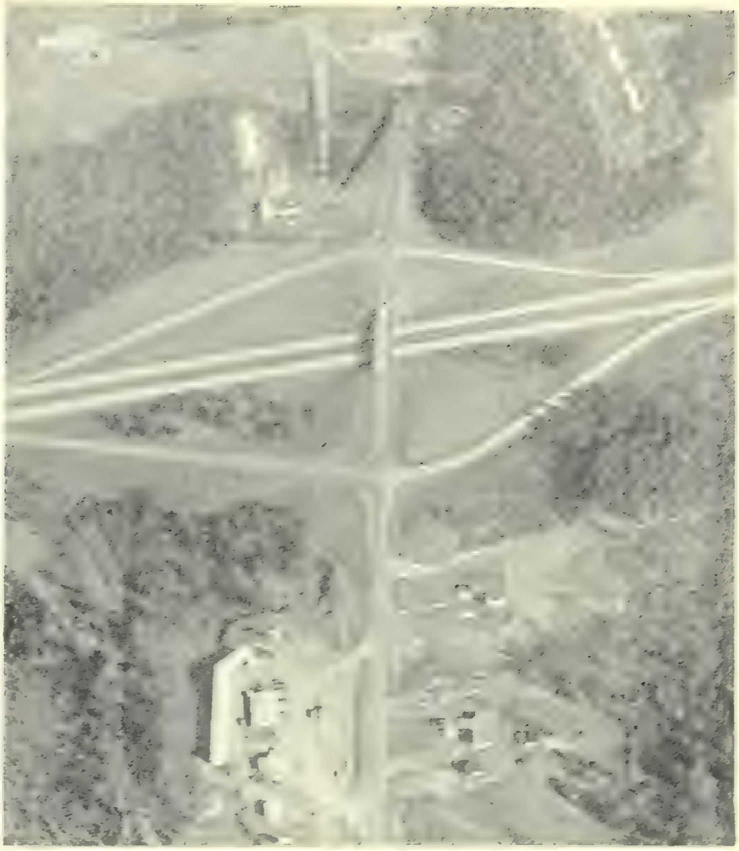

Highway-oriented land use at interchange areas is illustrated by these before-after photographs of the interchange near Lincoln, Alabama, where highways I-20 and U.S. 231 intersect. Photograph at left was taken in 1964, shortly after construction of the interchange was completed. In photograph at right, taken in 1968, new businesses—motel, 3 service stations, cafe, and fruit stand—that had located near the interchange are shown. Arrow points to most recent addition, a service station.

Highway Interchange Area Development—Some Recent Findings

Reported by
MARTIN M. STEIN, Economist
Economics and Requirements Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

By combining data on 332 interchanges in 16 States, the author has shown that interchange land development is affected both by the type of intersecting highway and by the relative accessibility of the interchange quadrants. According to his review of Alabama interchanges, a large part of development near predominantly rural interchanges consisted of highway-oriented businesses like motels, service stations, and restaurants. In contrast, a windshield survey of the Capital Beltway around Washington, D.C., disclosed that rapid growth in the numbers of apartment houses, churches, schools, shopping centers and industrial parks occurred at predominantly suburban interchanges subsequent to highway construction.

In other recent studies on interchanges, multilinear regression analysis and simultaneous linear models were used to predict interchange land development and to show that important factors of land development at an interchange are traffic volumes, distance to the nearest urban center, and populations of the nearest urban center.

Interchange research can assist highway planners and other officials to recognize potential interchange problem areas. The development of more precise analytical research tools and the formulation of a complete interchange planning process would facilitate the optimal economic development of highway interchanges and, simultaneously, protect their traffic capacities as vital components of an efficient transportation network.

A RELATIVELY large part of future intercity traffic is expected to use the Interstate Highway System, and many highway-related businesses can be expected to desire locations near interchanges. Industries that rely on motor carriers to obtain raw materials or distribute finished products are likely to seek plant sites close to Interstate highways. Even though interchanges in urban areas are spaced only about a mile apart, demand for the abundant nearby land is increased by the many land uses—commercial businesses, apartment houses, and institutions—that are attracted to interchange locations.

The many economic and social activities that locate near interchanges create a planning[1] problem. As one function of interchanges

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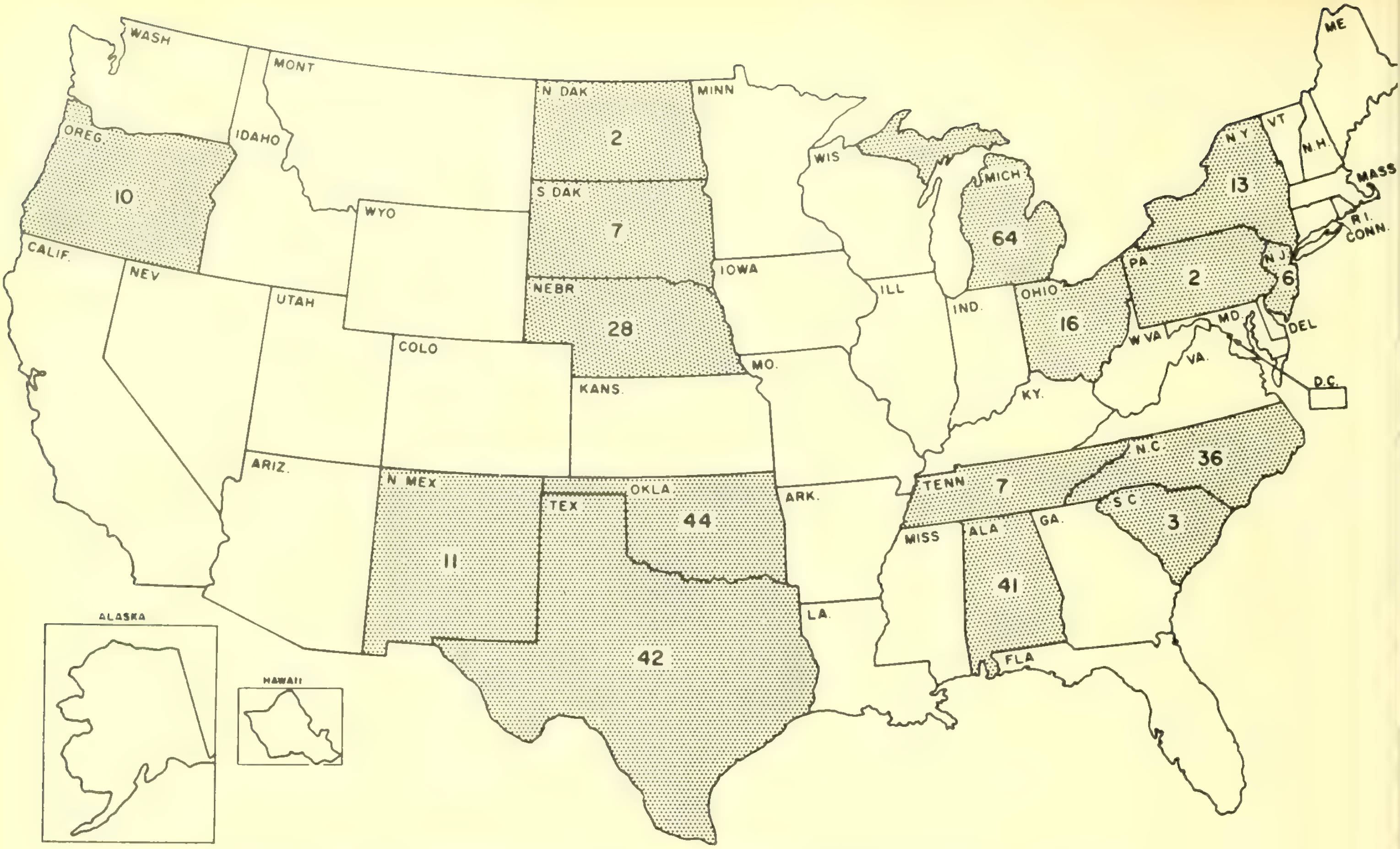
[1] Planning is used here to describe the highway planning and design process that relates to specifying interchange design type, location, and design volume, and to effective land-use and traffic-control techniques to maintain design capacity.



TOTAL NUMBER OF INTERCHANGES -- 332. DATA OBTAINED FROM INTERCHANGE LAND-USE STUDIES AND PUBLIC ROADS SEVERANCE DATA.

Figure 1.—Distribution of interchanges surveyed, by State.

Table 1.—Interchanges in survey, by year of survey[1]

Year of survey	Interchanges in survey	
	Number	Percent
1960	5	1
1961	15	4
1962	23	7
1963	24	7
1964	119	36
1965	19	6
1966	115	35
1967	12	4
Total interchanges in survey	332	100

[1] Date of construction for each interchange is not available.

is the provision of access to cross routes, the increased economic and social activity often produces traffic that exceeds original design estimates, and a spiral of activity develops possibly to impair freeway operation or cause serious traffic congestion on cross routes. Naturally, because interchange traffic consists of turning movements, any traffic congestion not only will increase accident potential near the interchanges, but also will impose another form of economic cost to the highway user—increased travel time. Each interchange represents a large investment, and careful planning is needed to maximize potential economic benefits to the community by providing locations with improved accessibility while minimizing impairment to efficient highway operations.

Interchange Land Development

Bureau survey of 332 interchanges

An awareness of potential problems at interchanges has prompted studies in nearly half the States (*1*).[2] Several of these studies and some severance studies, both of which describe interchange land use, have been combined to isolate general characteristics of land development and economic activity near interchanges. In the survey reported here all recent studies that contained information about interchange land uses were considered.[3] The distribution by State of the 332 interchanges surveyed in this manner is shown in figure 1, and the year that each interchange was surveyed is given in table 1. Aerial photographs and graphic and verbal descriptions were used to subjectively evaluate each interchange quadrant to determine predomi-

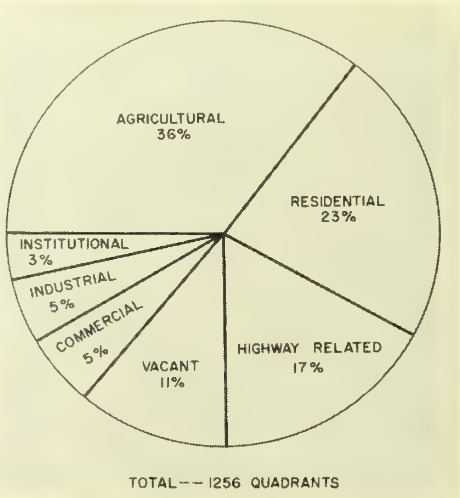


Figure 2.—Proportion of predominant land uses by quadrant—based on Public Roads survey of 332 interchanges in 16 States.

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[2] Italic numbers in parentheses identify the references listed on page 249.

[3] Except for a few interchanges in urban areas, the interchanges surveyed in the study reported here were in rural areas, and an analysis of urban-interchange characteristics would not yield statistically significant findings.

nant land use within a one-half mile radius of the interchange center. For example, in this analysis, a motel, service station, and restaurant in a quadrant would be considered predominantly highway-related land uses, whereas a school and a church in a quadrant would be considered predominantly institutional land uses. Shopping centers and other retail outlets were classified commercial uses, and factories and warehouses industrial uses. As shown in figure 2, agricultural, residential, and highway-related land uses were the three types of land uses most frequently recorded. The vacant category represents land that was being held for speculation, was undergoing construction, or did not exhibit evidence of economic activity.

Researchers compared general land development according to the types of highways forming the interchanges. Interchanges formed by the intersection of non-Interstate highways, such as a U.S.-numbered highway and a State-numbered highway, were found to have a much greater proportion of commercial, industrial, and institutional land uses, possibly because Interstate highways were constructed more recently and have not yet attracted as many of these activities, or because different design standards were used for non-Interstate interchanges. As data on construction dates were not available for all interchanges, the time factor could be considered only for the Capital Beltway and Alabama analyses to be discussed later. As shown in table 2, non-Interstate interchanges had more than twice as many commercial, industrial, and institutional quadrants as those with at least one Interstate highway.

According to individual evaluations highway-oriented land uses did not locate randomly at the various types of interchanges (table 3). Twenty-eight percent of the motels were located near interchanges of two non-Interstate highways, which represented only 4 percent of the total number of interchanges in the survey. In contrast, 32 percent of the motels had locations near interchanges of Interstate highways and local or county roads, although these interchanges represented 50 percent of those in the survey. Moreover, 28 percent of the service stations were located near interchanges of Interstate and U.S.-numbered highways, even though this type of interchange comprised 21 percent of the total number of interchanges in the survey. Other tourist-oriented businesses, like souvenir stands, also seemed to prefer Interstate and U.S.-numbered highway interchanges, as 56 percent of those surveyed occupied sites near them. Preliminary statistical analysis has indicated that these variations are significant at the 95-percent level of confidence.

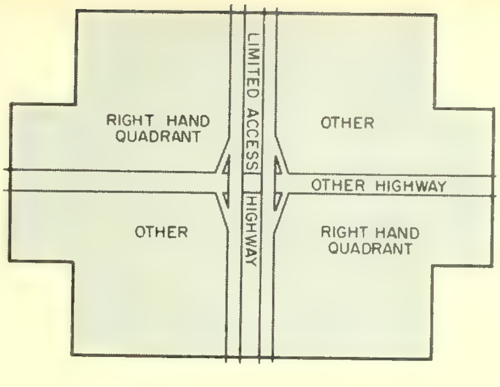

Figure 3.—Interchange quadrants

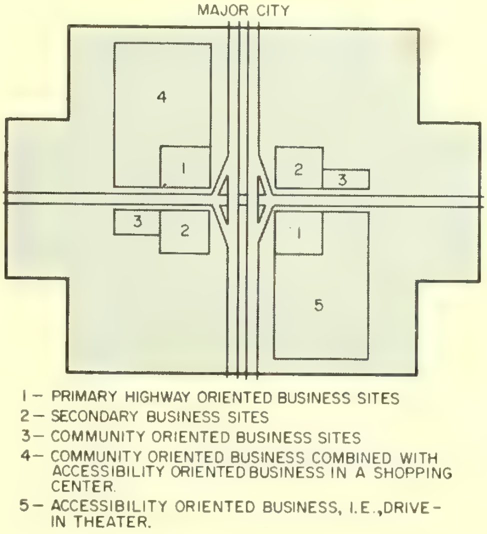
MAJOR CITY
4
1
2
3
3
2
1
5
1 — PRIMARY HIGHWAY ORIENTED BUSINESS SITES
2 — SECONDARY BUSINESS SITES
3 — COMMUNITY ORIENTED BUSINESS SITES
4 — COMMUNITY ORIENTED BUSINESS COMBINED WITH ACCESSIBILITY ORIENTED BUSINESS IN A SHOPPING CENTER.
5 — ACCESSIBILITY ORIENTED BUSINESS, I.E.,DRIVE-IN THEATER.

Figure 4.—Interchange development, preferred quadrants—adapted from Ohio report (4).

Table 2.—General land development at interchanges, by highway classification
[Public Roads analysis of 1,256 interchange quadrants in 16 States]

Type of highway interchange	Interchange quadrants	Type of land development			
		Agricultural and/or vacant	Residential	Commercial, industrial, institutional	Total
	Number	*Percent*	*Percent*	*Percent*	*Percent*
Interstate highway—local road	628	43	27	30	100
Interstate highway—State-numbered highway	296	60	14	26	100
Interstate highway—U.S.-numbered highway	260	48	26	26	100
Non-Interstate highway—non-Interstate highway	52	15	23	62	100
Interstate highway—Interstate highway	20	55	25	20	100
Total	1,256	....	....	... .	...

Table 3.—Distribution of highway-related land uses at interchanges, by highway classification
[Public Roads analysis of 332 interchanges in 16 States]

Type of highway interchange [1]	Service stations	Restaurants	Motels	Other tourist-oriented businesses
	Percent	*Percent*	*Percent*	*Percent*
Interstate highway—local road	49	48	32	44
Interstate highway—State-numbered highway	23	21	19	... ...
Interstate highway—U.S.-numbered highway	28	22	21	56
Non-Interstate highway—non-Interstate highway	.....	9	28	...
Total	100	100	100	100

[1] No highway-related land uses were recorded at interchanges of 2 Interstate highways.

Industrial or institutional land uses—factories, warehouses, and golf courses—were also disproportionately distributed at interchanges of two Interstate highways. For example, 17 percent of the warehouses and factories in the survey preferred this type of interchange, which was 2 percent of the total number of interchanges analyzed. All of the interchanges in this group were of the directional type and did not provide direct access to adjacent land; nevertheless, they can be attractive to those who seek a location near two major intercity highways.

Most preferred quadrants

For interchange planning, it is important to know the preferred location of highway services on the interchange. Several studies have indicated that the most desirable location for highway services is the first quadrant on the motorists right as he approaches the interchange (*2*, *3*)[4] (see figures 3 and 4). Right-hand quadrants ordinarily can be reached more easily than other quadrants and are prominently visible—two factors that are important to the location of highway-oriented businesses. Accordingly, the interchange quadrants in the survey were divided into two groups—right quadrants and other quadrants—to test the hypothesis that right quadrants are preferred by highway-related businesses. (A significant variation was noted here at the 90-percent level of confidence.) Although 59 percent of the quadrants containing predominantly highway-related businesses were right-side quadrants, only 43 percent of those containing predominantly industrially related land uses were. (see figure 5).

Quadrants that lack accessibility to the interchange attract fewer land uses that depend on motorists for existence. For this reason directional interchanges do not seem to attract the same proportion of highway-related services as other types of interchanges (fig. 6). Eleven percent of the directional

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[4] Unpublished paper presented to the North Central Land Economics Research Committee also provided land value data (see acknowledgments).

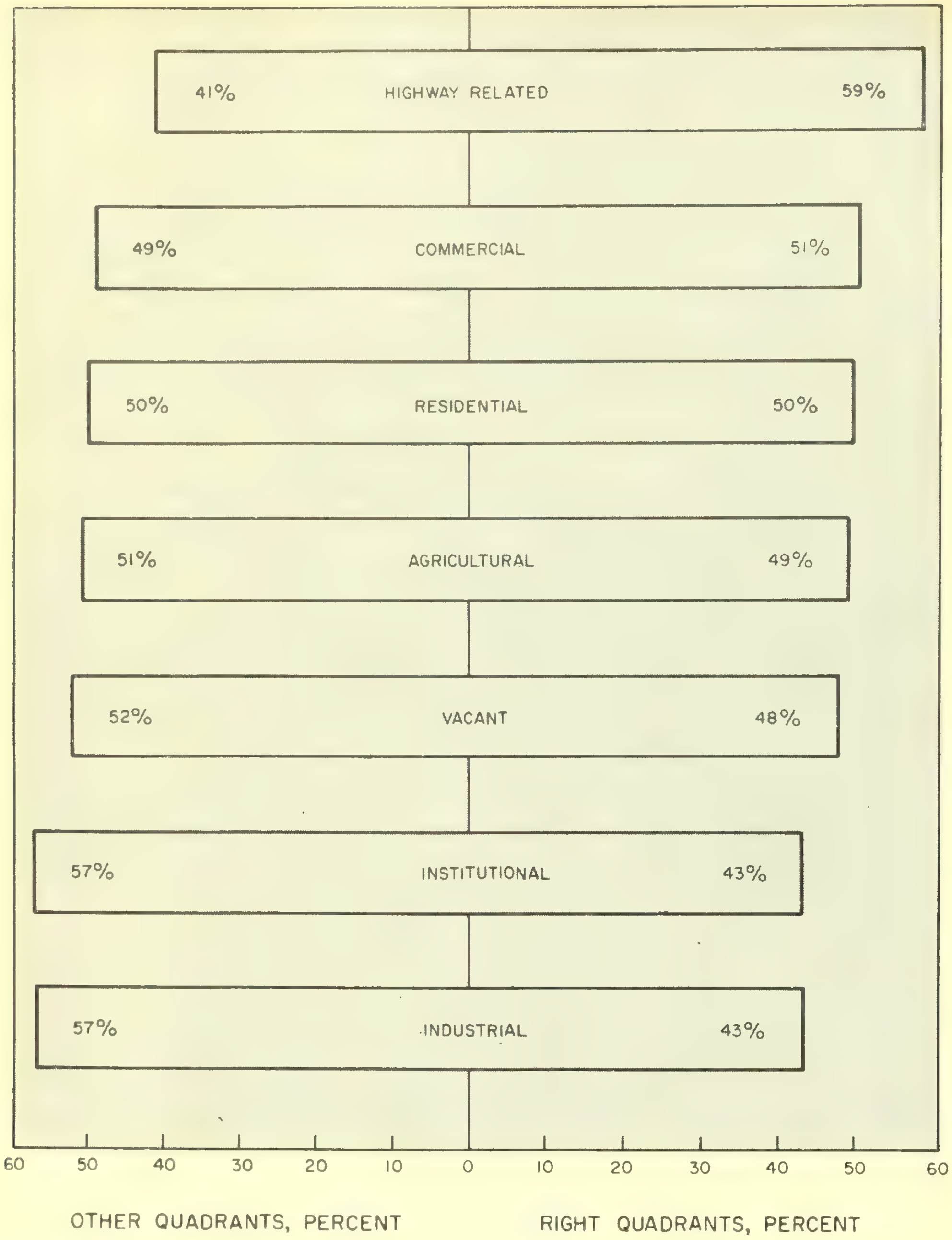

Figure 5.—Distribution of general land uses by quadrant—based on analysis of 1,256 interchange quadrants.

interchange quadrants surveyed were classified highway related whereas the proportion for partial cloverleaf interchange quadrants was 19 percent.

Free access and restricted access interchanges

The importance of access to the potential of land development is generally recognized. This access can be provided either by a free access crossroad or by a frontage road along the freeway. Frontage roads often provide good locations for highway-oriented services. Of the full-cloverleaf interchanges surveyed, 25 percent of the quadrants served by frontage roads were used for highway services. Interchanges without freeway or frontage-road access to the abutting properties—restricted-access interchanges—had no highway related uses. Other commercial uses also appear to gravitate to free-access interchanges (fig. 7).

Alabama interchanges

A good source for analysis of the growth of land uses near interchanges was provided by data on 57 Alabama interchanges (*4*), which showed interchange development from 1964 to 1968.

Land uses were recorded more frequently after highway construction had been completed (fig. 8). The greatest increase, in both absolute and relative terms, was in highway-related land uses. Although both commercial and industrial land uses exceeded highway-related uses before highway construction, highway-related uses were predominantly intensive in 1968, approximately five years after highway construction. As shown in figure 9, the increase in the highway-related land use was primarily a result of the increased number of service stations. In 1964, 27 percent of the 57 interchange areas had at least one service station. By 1968, the percentage had increased to 67 (fig. 10).

Predicting interchange land development

Data collected on land development at interchanges can be useful to both highway planners and local communities because it not only indicates the past economic growth, but also can be used to estimate future growth. One simple method of anticipating shifts in land use near interchanges is through observation. As shown in figure 11, eight Alabama interchanges that contained motels also contained a service station or a restaurant, or both. Often these uses occurred during motel construction or immediately after construction had been completed. As motels are usually associated with other highway-oriented businesses, interchange development can be predicted by noting the locations of motels or of their planned constructions. This may then serve to initiate a more thorough investigation of the interchange, which could include evaluation of travel time to the nearest urban center and of the center's population; traffic counts; and surveys of local officials, planners, businessmen, or developers.

Another method of predicting which interchanges will experience rapid development is to compare the proportions of specific interchange land uses that are occurring. Several ratios of service stations and restaurants to motels, are given in table 4. Some of the differences in these ratios may be due to the fact that motels locate more frequently near

Table 4.—Ratios of service stations and restaurants to motels at interchanges

	Service stations	Restaurants	Motels
	Ratio	*Ratio*	*Ratio*
Alabama [1]	8.5	1.6	1.0
Michigan: [2]			
Major city interchange	2.7	1.9	1.0
Secondary city interchange	9.2	3.1	1.0
Composite	4.3	2.4	1.0
Pennsylvania [3]	2.7	1.9	1.0
National [4]	2.7	1.0	1.0

[1] Ratios derived from Alabama report (*5*).

[2] Ratios derived from data collected in report, *Interchange Development Along 190 Miles of I-94*, Michigan State Highway Department, 1964.

[3] Ratios are estimates used in Pennsylvania report (*6*).

[4] Ratios derived from Public Roads survey of 332 interchanges.

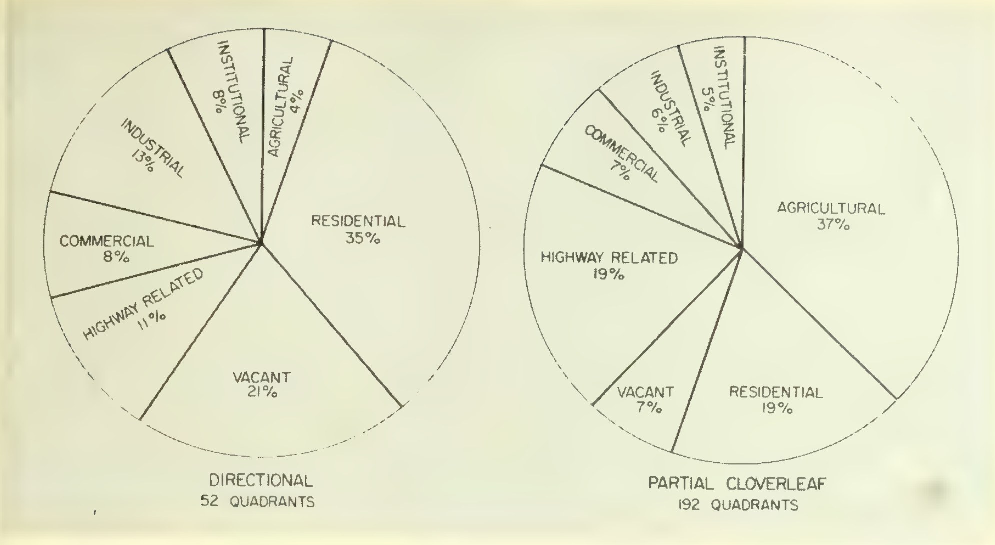

Figure 6.—Interchange land development by interchange design.

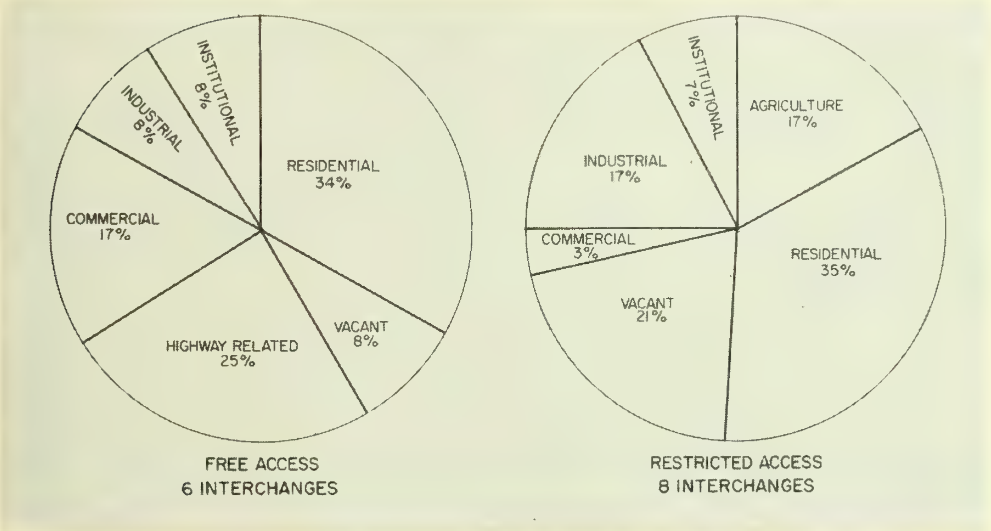

Figure 7.—Land development at full cloverleaf interchanges, by type of access—based on 14 full cloverleaf interchanges.

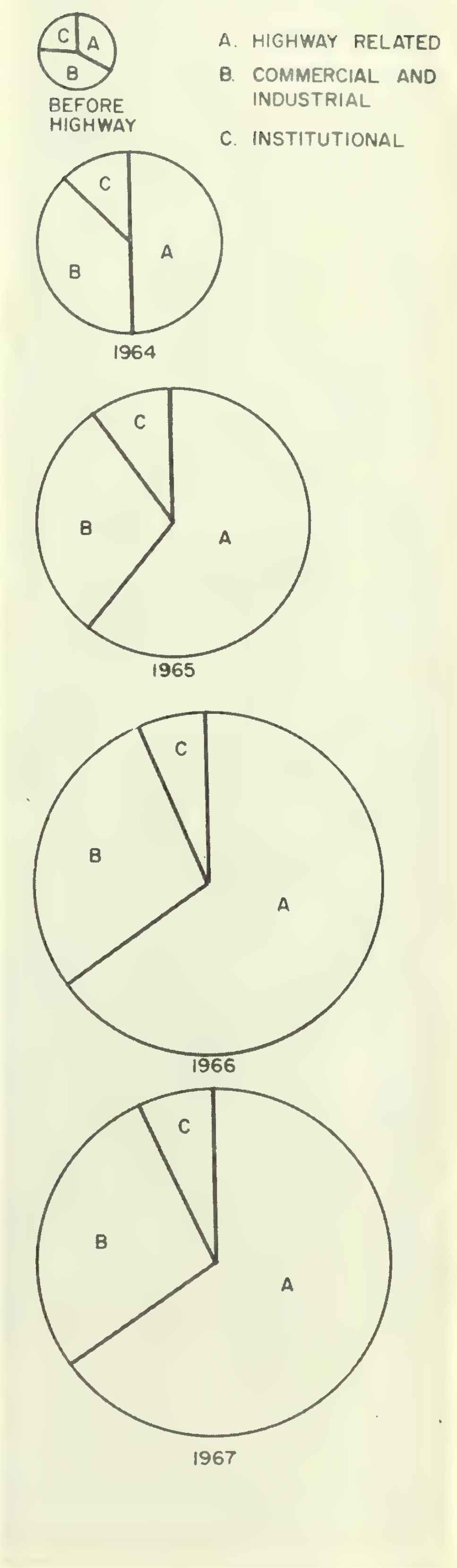

Figure 8.—Distribution of selected land uses at 57 Alabama Interstate interchanges. (Sizes portray growth in number of each of three uses in interchange area—not relative land area in use.)

areas of large population concentration and service stations more frequently in areas of sparse population.

The value of ratios of this type was shown by researchers of the Institute for Research on Land and Water Resources at Pennsylvania State University (*5*) who used them to help predict the economic impact of interchanges on the economy of Clinton County, Pennsylvania. The researchers predicted that four interchanges located in the county would result in the construction of a new 100-unit motel, two new restaurants, one trucking terminal, and four service stations, and that these new businesses would increase the county's total economic activity (estimated through the use of an input-output model) by $5 million, or 2 percent. This figure included a 3-percent increase in annual government receipts of $230,000.

A multilinear regression analysis performed in a study by the West Virginia University Center for Appalachian Studies and Development for the West Virginia State Road Commission (*6*) tested with selected variables the relation of the number of gasoline pumps, restaurant seats, or motel rooms at 79 interchanges in nine States. Factors considered in that study included the number of years the interchange had been open, distance from the interchange to the center of the nearest city, and the population of the nearest city. The strongest statistical relationship was found between motel rooms and the population of the nearest city.

Researchers at Pennsylvania State University continued the study of development at interchanges. In their report (*7*) they verified the feasibility of developing a predictive, interchange-development model of a

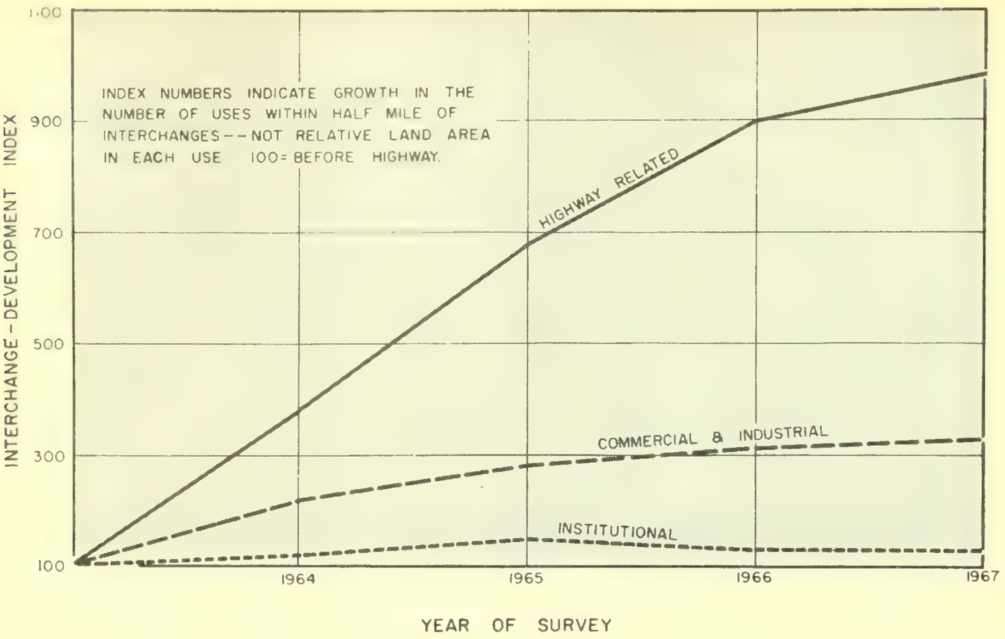

Figure 9.—Relative growth of selected land uses at 57 Alabama Interstate interchanges.

simultaneous-equation type. This model further developed in a doctoral thesis by Richard D. Twark at Pennsylvania State University in 1967, is described in a later progress report (*8*). A two-stage, least-squares technique was used in the thesis to estimate statistical parameters of the structural equations that describe interchange development at 105 interchanges. The researchers determined that average daily traffic on cross routes was the single most important factor in explaining new highway-oriented development. Other researchers at Pennsylvania State University developed a model that simulated spatial diffusion of land uses at interchanges and considered threshold size of population needed to support a given function (*9*).

Similarly, in a study of Oklahoma interchanges (*10*), traffic volumes and size and distance of the nearest urban area were noted to be primary factors in the development of highway-service-oriented establishments. Detailed land use maps were used to describe land development near interchanges. Aerial photographic observation, land-use maps, and simple charts provide a handy combination to evaluate interchange development. More sophisticated techniques can be implemented as the level of economic-development analysis warrants more complex analytical tools.

Interchange Land Development at Circumferential Highways

Capital Beltway, Washington, D.C.

Circumferential highways or beltways around urban areas have different characteristics from other types of limited-access highways. Even though one of their main functions is to divert intercity traffic, the origin or destination of which is not in urban areas, they serve many suburban residents and businesses.

The Capital Beltway, Interstate Highway 495, is a 66-mile freeway that encircles the most densely populated part of the Washington metropolitan area. Since its completion, several studies have been conducted to determine the impact of this highway on surrounding socio-economic development. In one study, a University of Virginia study (*11*), it was determined that nearly 48 percent of industrial employment in Northern Virginia was located within a mile and a half of the Beltway. According to officials of recently located firms, the Beltway was a prime factor in their site-selection decisions. It was noted in the study that since the Beltway was opened, more than 3,000 apartment units were built within a mile and half of the Beltway. When apartment residents were sampled, 50 percent stated that they used the Beltway to go to and from work.

In another study (*12*) conducted on the Maryland portion of the Capital Beltway, similar economic attraction was discovered. The researchers in this study noted that the number of recreation trips between Virginia and Maryland suburbs increased after the Beltway had been constructed. Some of the social aspects of beltway-effect were studied in a Public Roads survey of churches that were located within a half mile of beltway interchanges (*13*), in which it was determined that many churches had located near the Beltway to take advantage of the improved accessibility and visibility provided by a beltway location (fig. 12).

After the Beltway was constructed, annual windshield surveys were conducted annually by the Bureau of Public Roads to provide an additional perspective of the beltway-effect on land development. Even though the Capital Beltway was completed in 1964, only 24 of nearly 40 interchanges had been opened, and data on land use recorded for these interchanges were compared with subsequent surveys. Selected land uses at the interchange quadrants since 1964 are shown in figure 13. Churches were found at more Beltway interchange quadrants than any other type of land use. The next most frequent land uses, in order, were apartments, schools, and shopping centers. In contrast to the Alabama survey, land uses for highway-oriented businesses were not predominant. Apparently, the tendency for residentially oriented land uses, such as apartments, schools, and shopping centers, to locate at interchanges of circumferential highways may increase.

Because circumferential highways surround well-developed, populated areas, land uses adjacent to the highway will reflect the pattern outlined in the preceding paragraph. Apartments and churches can be located near other residential areas and are more dispersed than the highway-oriented businesses that tend to

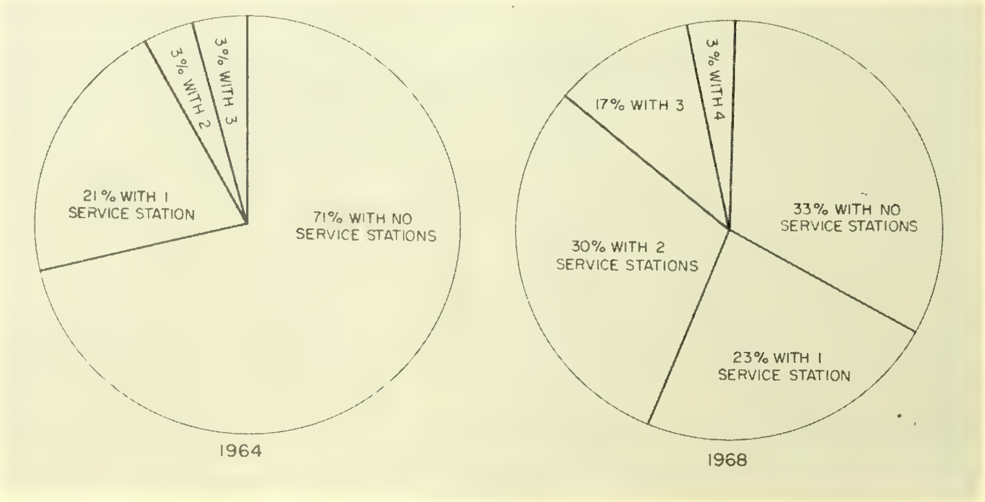

Figure 10.—Distribution of service stations at 57 Alabama interchanges, 1964–68.

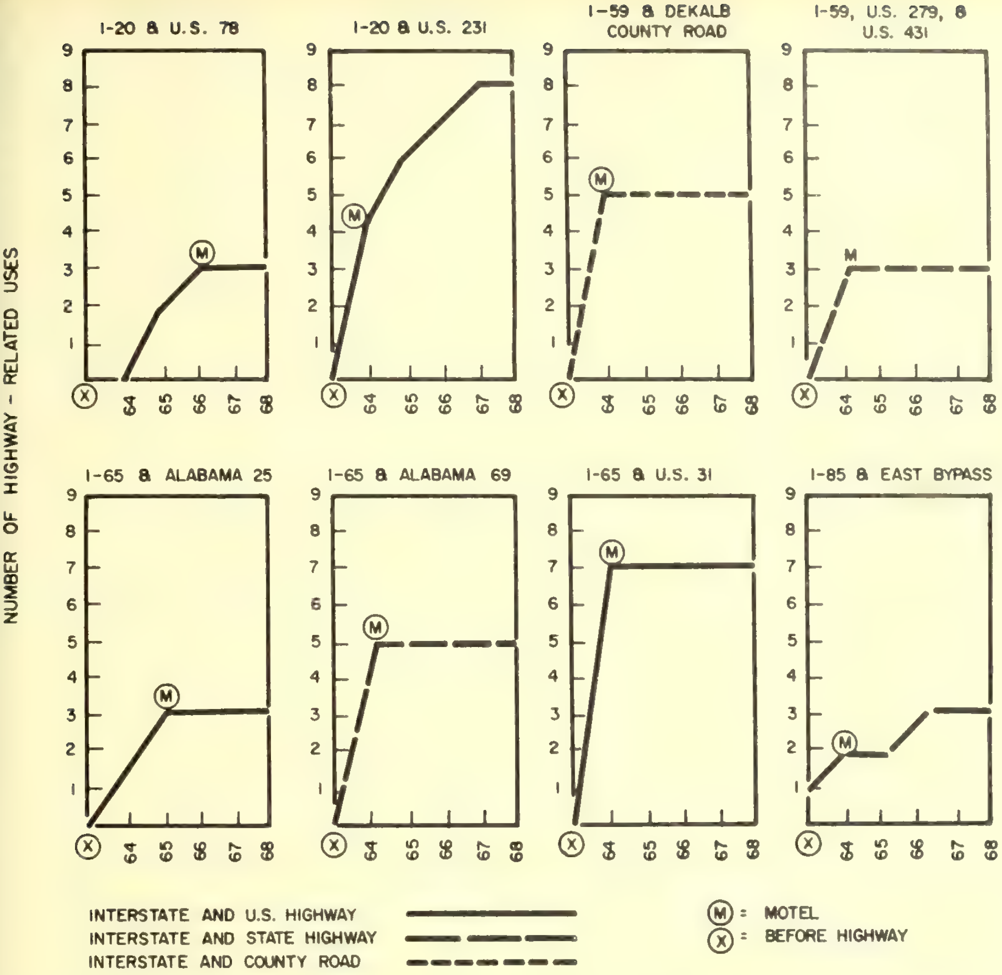

Figure 11.—Growth of highway-oriented uses at 8 selected Alabama interchanges—a profile of development.

prefer certain interchange areas that are most accessible to through traffic.

Another measure of development at interchanges is the number of driveways in each quadrant. Figure 14 shows a shift from quadrants with fewer than four driveways to quadrants with more than four driveways. Thus without land controls, the number of driveways connecting to feeder roads increases as demand for land at interchanges increases.

The Capital Beltway is only one example of the attraction that highway improvements have on economic development. But beltways around large urban centers are different from the usual highways because they attract residential, industrial, and commercial development. Thus the growth in the numbers of apartment projects, shopping centers and industrial parks at beltways is more rapid than highway-oriented land uses because of the proximity of large population centers.

Other beltways

The Massachusetts Beltway, Interstate Highway 495, which serves the Boston metropolitan area, was also analyzed (*14*) for its effect on police and fire-protection services. Researchers determined that this Beltway facilitated mutual-aid calls between suburban police and fire departments even though it caused an access problem when fires occurred between interchanges. They noted that traffic in local towns was reduced after completion of beltway construction because through travelers bypassed town streets.

The effect of this highway, also known as *Platinum Pike*, *Loot Route*, and *Silver Scimitar*, was discussed by Governor John A. Volpe, now Secretary of Transportation, in a seminar at Dean Junior College, Franklin, Massachusetts, in 1965 (*15*). The purpose of the seminar was to evaluate the impact of the beltway and determine the best way to maximize its potential contribution to the socioeconomic development of the area. One reason for interest in Interstate Highway 495, was the startling development along Massachusetts Route 128 where more than 340 industrial plants, with an annual payroll of more than $400 million, had located by 1965. Development along Route 128, once called the *Route to Nowhere*, was also studied by the Massachusetts Institute of Technology in a 1958 study (*16*).

In 1960, a study was conducted on portions of the Baltimore Beltway, Interstate Highway 695, by the Maryland State Roads Commission (*17*). According to the researchers, residents mentioned increased privacy and accessibility as advantages of beltway proximity.

In the Boston and Baltimore Beltway studies it was shown that land development increased near circumferential highways. Because these highways often are located in more densely populated areas than rural Interstate highways, they apparently experience different patterns of land development, which should be considered in future techniques for interchange research, planning, and control. Some possible land uses at beltway interchanges are commuter parking, mass-transit terminals, and motorist or tourist information centers. Research on one of these possibilities is now in progress to consider the problems caused by commuter parking at interchanges.

Use of Interchange Research

Several States have attempted to implement the results of interchange-land-development analysis by formulating interchange controls. It was suggested in a report (*18*) by the Utah State Department of Highways that the installation of parks at interchange areas may help maintain interchange transportation efficiency, as well as provide recreational opportunities, and promote tourist trade. Recently, a proposal to establish a recreation-beautification project in the interchange area of the Dan Ryan and Stevenson Expressways was discussed at a meeting in Chicago, which was arranged by the Chicago Federal Executive Board and attended by representatives of State, municipal, and Federal agencies—Bureau of Public Roads; Federal Highway Administration; Department of Housing and Urban Development; Department of Health, Education, and Welfare; and the Department of the Interior.

In a report for the Illinois Department of Highways (*19*) it was suggested that State highway departments expand the role of interchange planning by coordinating this function with other community planning agencies and urban renewal programs, and thereby establish guidelines to assist developers and planners when land development near interchanges is being considered. As implied in figure 15, the intensity of planning will vary with the intensity of development. Additional control techniques can be adopted when warranted by projected development. Land development-projection techniques facilitate the recognition of potential problem areas.

Recommendations for Further Research

Many research techniques have been used to analyze land development at interchanges. Nearly 100 studies provide data on land development. The consolidation of several studies using some elementary statistical methods to present a nationwide summary of land development at interchanges has been attempted in the study reported here. The fact that some significant patterns of land development were found may indicate the need for more detailed and sophisticated research approaches, in which these additional factors should be considered:

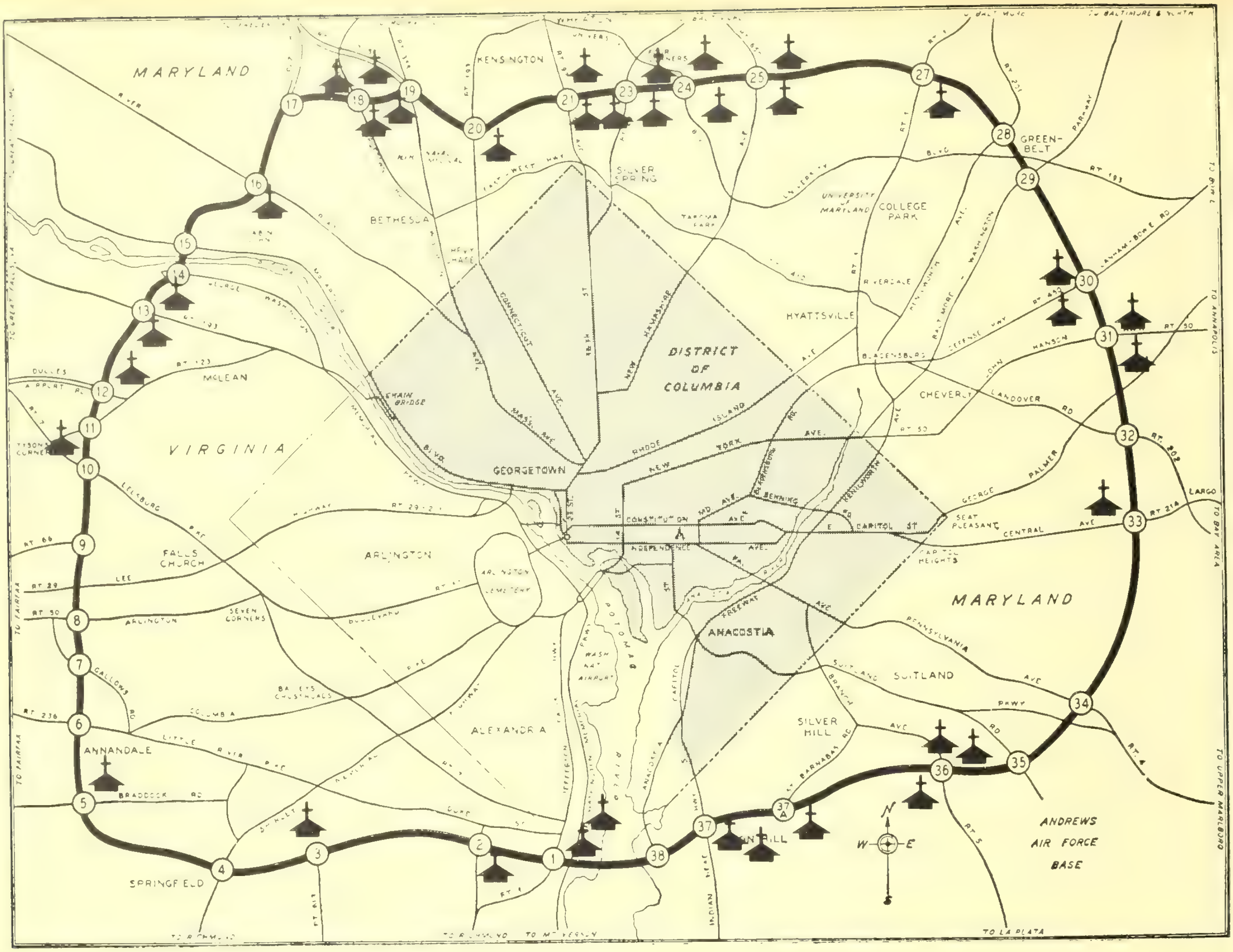

Figure 12.—Locations of 36 churches near interchanges of Capital Beltway, Washington, D.C. 1967.

- Interchange land development may differ according to type of interchange design and type of limited-access facility concerned.
- Land development at interchanges may be related to initial catalytic land uses that attract other related land uses.
- Interchange land-development research should not be separated from the interchange planning and control process problem.

Generalizations about interchanges can be made if the data are properly organized before the analysis is conducted. The studies that have group interchange land data in the early stages of tabulation produce the most significant findings. Analysis of interchange quadrants and specific land uses also reduces the level of aggregation and permits more careful analysis, which in turn, helps to increase the significance of the findings.

The appearance of a specific catalytic agent in land development is not evident immediately unless land development is measured over a period of time. Land-use planners generally recognize the ability of certain land uses to generate or attract other land uses.

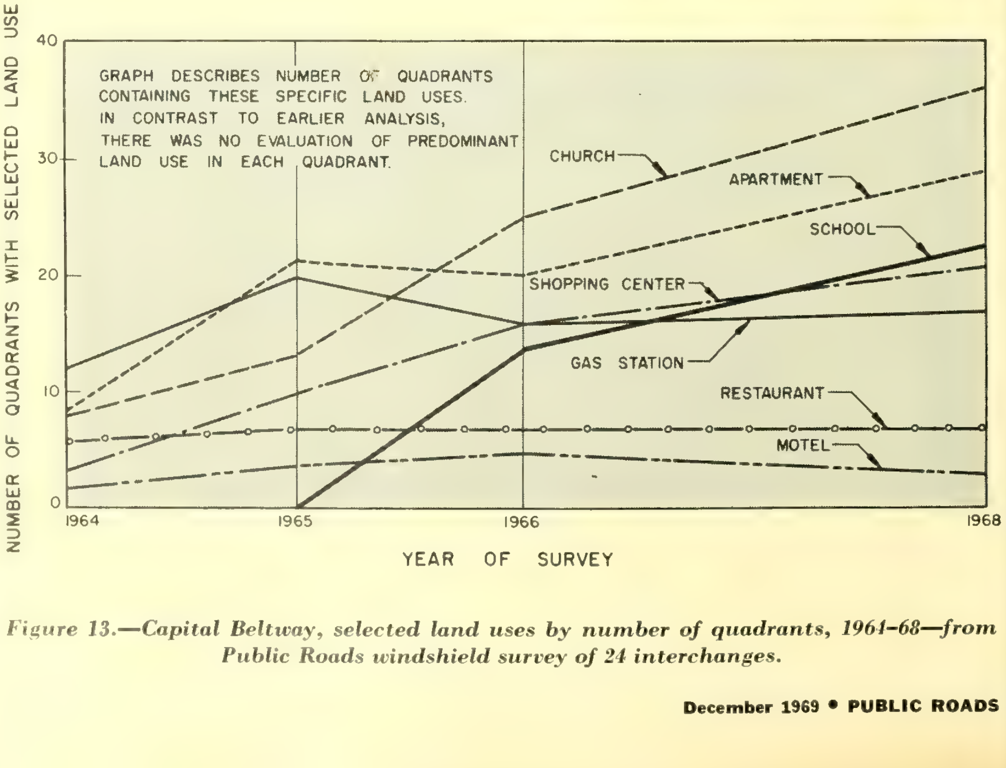

Figure 13.—Capital Beltway, selected land uses by number of quadrants, 1964–68—from Public Roads windshield survey of 24 interchanges.

Land near interchanges seems to experience catalytic growth as shown in the foregoing analysis of land development at Alabama interchanges. Future interchange research might find this type of growth to be responsible for some interchange development patterns. If highway planners can predict which interchanges will attract the catalytic form of land use, they could design the interchanges to handle additional traffic volumes or apply land-use controls to reduce traffic pressure. A dynamic model of economic development at interchanges may be one approach to the solution of this problem.

Analysis of interchange land development must be coordinated with interchange planning to permit maximum use of research findings. Research findings can be useful to both highway planners and local communities by indicating which interchanges will experience rapid land development. Because rapid land development is often associated with increased trip generation, highway officials can take immediate steps to preserve the efficiency of the interchanges as a transportation investment by considering possible design alterations to improve interchange traffic capacity.

On the other hand, local planning officials may also wish to take land-use control measures that limit the density of land development near the interchange. Coordination of both planning functions, land development and interchange planning, is required to permit optimal economic development and to avoid interchange traffic congestion, which not only creates a safety hazard but also reduces the efficiency of the entire highway network. The chart shown in figure 15 describes some of the factors that may be part of this process.

Summary

Land-use data at 332 interchanges in 16 States have been analyzed to determine land development patterns at interchanges. Land development at these interchanges was determined to be largely agricultural and residential, although more than 17 percent of the interchange quadrants had predominantly highway-related land uses—motels, restaurants, and service stations.

Although the majority of interchanges studied were those with at least one intersecting Interstate highway, the major land uses at non-Interstate interchanges were commercial, industrial, and institutional. The concentration of these uses at non-Interstate intersections may be influenced by the relatively recent construction of the Interstate System and may indicate that greater land development will occur at Interstate interchanges in the future.

Certain types of interchanges were observed to contain significantly larger proportions of certain land uses. Land uses generally associated with residential development were predominant near interchanges of Interstate highways and local roads. Highway-related land uses tended to concentrate near interchanges of Interstate highways and U.S.-numbered highways. In contrast, interchanges of two Interstate highways contained relatively larger portions of industrial land uses. Perhaps this was caused by the increased regional accessibility of the sites or by the lower land values resulting from the relative lack of access to nearby developments.

At Alabama Interstate-Highway interchanges, land development within a radius of one-half mile from the interchange centers consisted largely of highway-oriented businesses. Interchanges near population centers, such as beltway interchanges, included larger proportions of higher land uses, such as apartment developments and industrial parks.

Studies of interchange development can help planners predict which interchange areas will experience unusually rapid development. The coordination of interchange research and highway planning facilitates formulation of control techniques and planning methods that preserve the efficiency of the interchange and promote optimal economic benefits to the community.

ACKNOWLEDGMENTS

The author acknowledges the assistance and guidance of the Public Roads staff who contributed to this article. Floyd I. Thiel, Preston J. Moe, and Edward M. Nolan conducted the windshield survey of the Capital Beltway and designed the interchange analysis forms. Merritt D. Nolden and Edna Wolf helped prepare the charts and tables and performed the statistical calculations on which this report was based. Miss Ingrid Farquarson performed a thorough literature search of interchange studies.

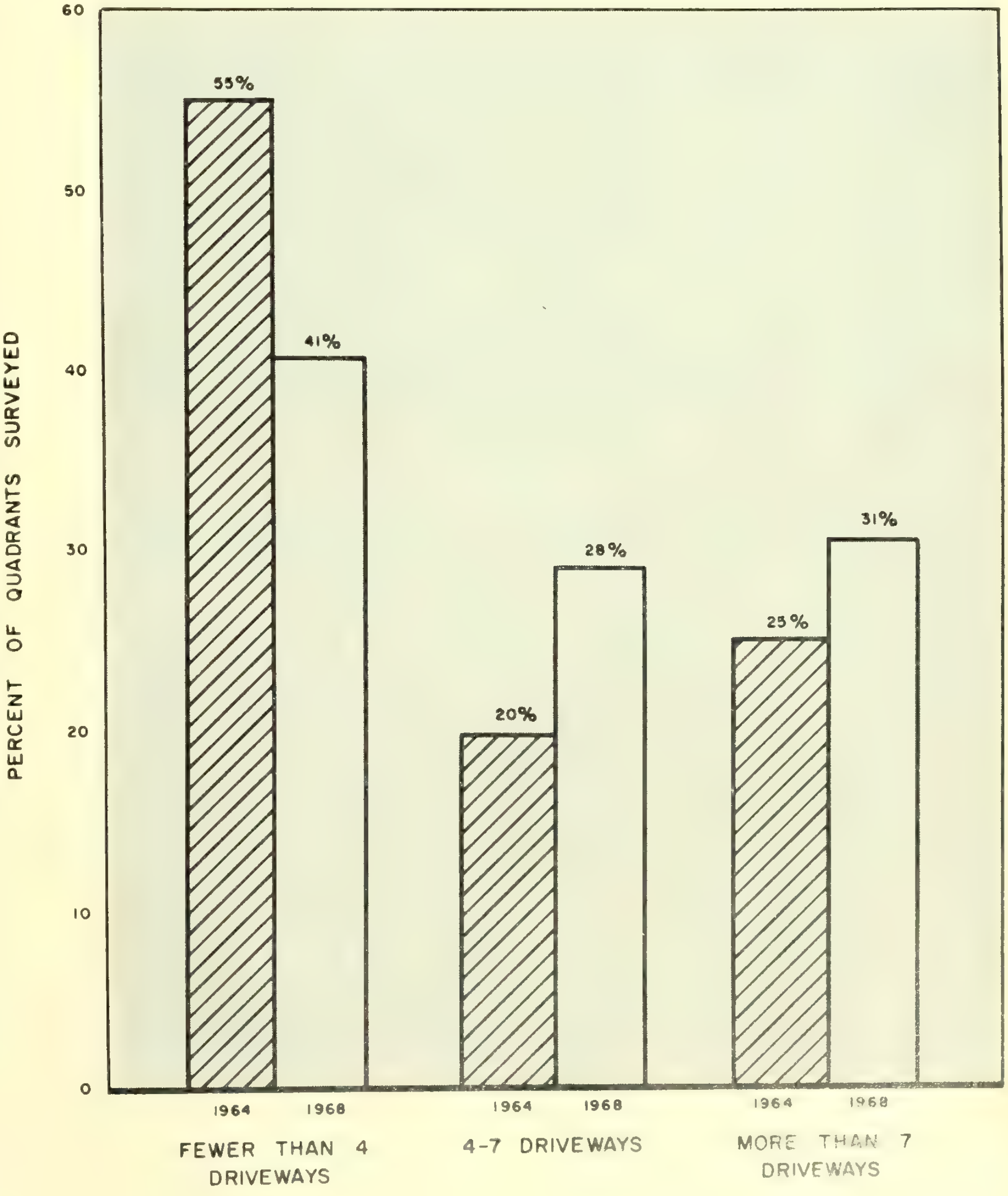

Figure 14.—Capital Beltway interchanges, number of driveways by quadrant, 1964–68—from Public Roads windshield survey of 24 Capital Beltway interchanges

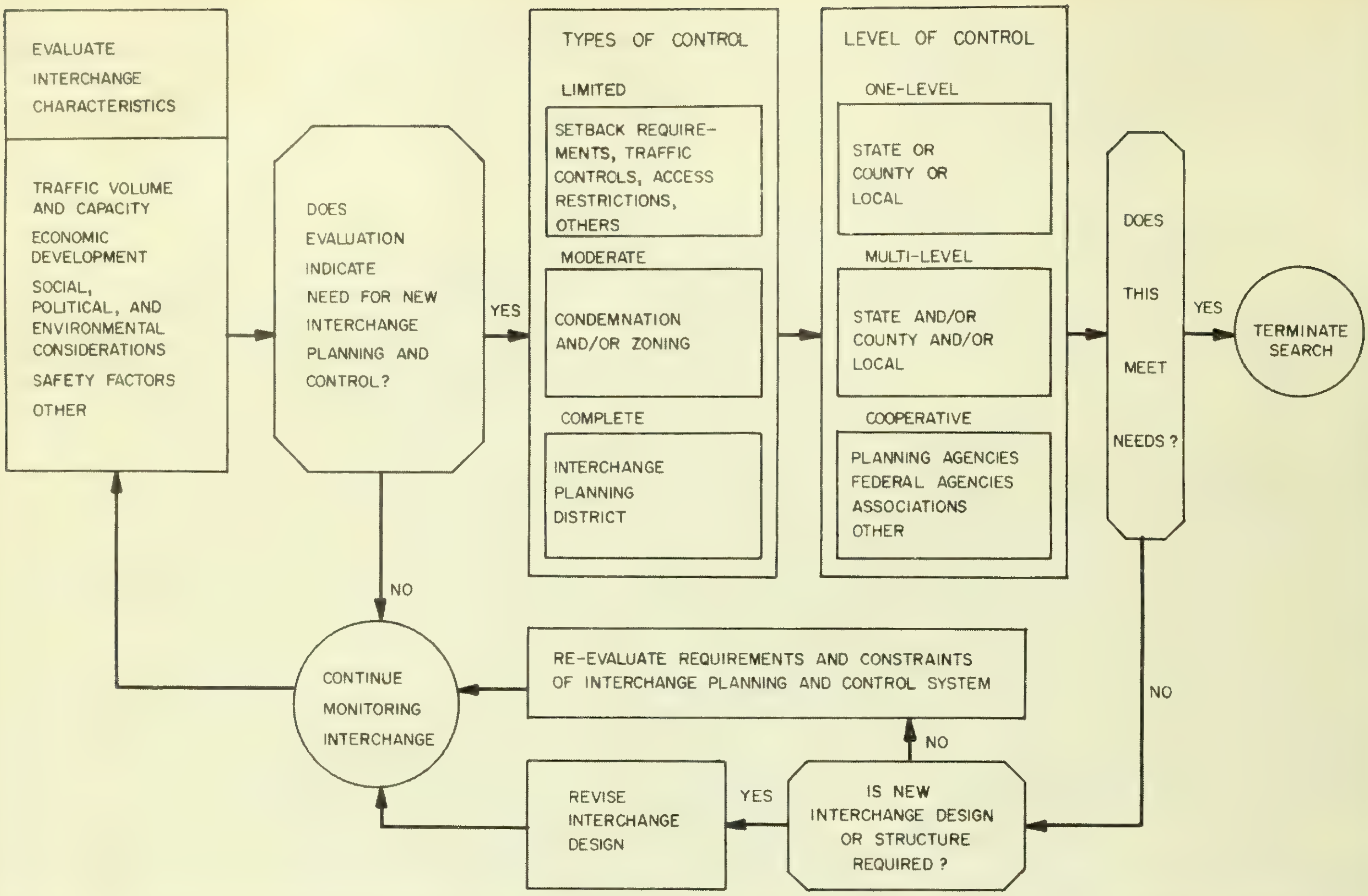
EVALUATE INTERCHANGE CHARACTERISTICS
TRAFFIC VOLUME AND CAPACITY
ECONOMIC DEVELOPMENT
SOCIAL, POLITICAL, AND ENVIRONMENTAL CONSIDERATIONS
SAFETY FACTORS
OTHER
DOES EVALUATION INDICATE NEED FOR NEW INTERCHANGE PLANNING AND CONTROL?
YES
NO
TYPES OF CONTROL
LIMITED
SETBACK REQUIREMENTS, TRAFFIC CONTROLS, ACCESS RESTRICTIONS, OTHERS
MODERATE
CONDEMNATION AND/OR ZONING
COMPLETE
INTERCHANGE PLANNING DISTRICT
LEVEL OF CONTROL
ONE-LEVEL
STATE OR COUNTY OR LOCAL
MULTI-LEVEL
STATE AND/OR COUNTY AND/OR LOCAL
COOPERATIVE
PLANNING AGENCIES
FEDERAL AGENCIES
ASSOCIATIONS
OTHER
DOES THIS MEET NEEDS ?
YES
TERMINATE SEARCH
NO
CONTINUE MONITORING INTERCHANGE
RE-EVALUATE REQUIREMENTS AND CONSTRAINTS OF INTERCHANGE PLANNING AND CONTROL SYSTEM
NO
REVISE INTERCHANGE DESIGN
YES
IS NEW INTERCHANGE DESIGN OR STRUCTURE REQUIRED ?

Figure 15—Suggested system for interchange planning and control.

The author also wishes to acknowledge the following unpublished paper, which provided background material for a part of this article: *Bureau of Public Roads Bank of Land Value Data—Its Status and Usefulness for Highway Related Developments*, by Floyd I. Thiel, Ruth B. Ross, and John Yasnowsky, presented to the North Central Land Economics Research Committee, Chicago, Illinois, 1965.

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(*3*) *Interchange Area Development*, by the H.O.P. Committee of Ohio, representatives from the Ohio Department of Highways, the Oil Industry, and the [illegible] Conference, Columbus, Ohio, 1963.

(*4*) *Photographic Comparison of Land [illegible] Areas Adjacent to Interchanges [illegible] Interstate System*, Highway Plannin [illegible] search Reports 8, 23, 30, and 36, by J. H. David, sponsored by the Alabama State Highway Department in cooperation with U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, 1964, 1965, 1966, 1967, and 1968.

(*5*) *The Impact of Interchange Development on the Economy of Clinton County*, by Hays B. Gamble, et al., Institute for Research on Land and Water Resources, Pennsylvania State University, 1966, p. 16.

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(*7*) *Factors That Influence Economic Development at Non-Urban Interchange Locations*, Research Report No. 9, by Owen H. Sauerlender, Robert B. Donaldson Jr., and Richard D. Twark, Institute for Research on Land and Water Resources, Pennsylvania State University, 1966, pp. 43–47.

(*8*) *A Predictive Model of Economic Growth at Non-Urban Interchange Sites on Pennsylvania Interstate Highways*, by Richard D. Twark, and Owen H. Sauerlender, Institute for Research on Land and Water Resources, Pennsylvania State University, 1965.

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(*Continued on p. 264*)

Traffic Marking Materials— Summary of Research and Development

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Principal Research Chemist,
Materials Division**

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Recent research in traffic marking materials is presented in this review to summarize the state-of-the-art since the last comprehensive bibliography published in 1952 by the Highway Research Board. It is intended not to present a complete bibliography on the subject since 1952, but rather, to point out the most pertinent research findings and developments in recent years. Research and development in the United States sponsored jointly by the State highway departments and the U.S. Bureau of Public Roads is emphasized and supplemented by information on other ongoing research by other agencies in the United States and throughout the world.

Many proprietary trade products are specifically mentioned to present a more complete account. Mentions of trade names are not to be considered endorsements of the products, nor are omissions of trade products, either overlooked or unknown to the author, to be considered inferences that such products are of lesser quality than those mentioned.

AS MOST highway engineers realize, traffic marking has become a sizable item in the maintenance budgets of many highway departments. In a published survey (*1*)[2], it was shown that throughout the United States, in 1965, 12 million gallons of conventional traffic paint were purchased by the States and an estimated additional 9½ million gallons by cities and counties. Thus city, county, and State governments nationally used more than 21 million gallons of conventional traffic paint. At average costs of $1.66 per gallon for white paint and $1.95 for yellow paint (*1*), the national cost for paint binder in 1965 alone was estimated at $38 million. It was also reported in the survey that approximately 107 million pounds of reflectorizing glass beads were purchased by these governments during the same year—66 million pounds by the States and an estimated 41 million pounds by cities and counties—at an estimated cost of about $12 million. Accordingly, the total cost of paint and beads during 1965 was approximately $50 million.

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[1] Presented at the 55th Annual Meeting of the American Association of State Highway Officials, Philadelphia, Pa., October 27–31, 1969.

[2] Italic numbers in parentheses identify the references listed on page 256.

But these material costs are reported to be only about a third of the total cost of installed conventional striping (*2*). Thus the annual overall cost of striping with conventional paint in the United States is estimated to be at least $150 million. This estimate does not include costs of other marking systems like hot thermoplastics, preformed stripes, or raised button-type markers, which are known to be appreciable. Not only these exorbitant costs, but also the need for adequate lane delineation to enable safe driving during all seasons of the year and in inclement weather, have stimulated pavement-marking research and development. The significant trends and developments in this field since the last comprehensive bibliography on pavement marking materials, published by the Highway Research Board in 1952 (*3*), is summarized in this review, which is intended not as a complete bibliography, by any means, but as a current state-of-the-art report to point up to present researchers and potential producers the most pertinent research and development in recent years.

In gathering information for this review, research and development in this country, particularly through Federally-aided highway studies (*4*) and similar experimental projects, was emphasized. In addition, information on recent international research, obtained through the Highway Research Information Service of the Highway Research Board, as well as available literature, reports, and unpublished information on domestic and international developments, were reviewed. Many proprietary products, which may not have been fully evaluated by unbiased consumers or governmental agencies, are specifically mentioned in this review. The mentioning of any product to provide the reader with a broader scope of developments is not to be construed as an endorsement of that particular product. Similarly, the names of many deserving trade products unknown to the author probably have been omitted.

Conventional Solvent-Based Traffic Paints

The main objectives of recent research and development in conventional traffic paints has been [illegible] durability and performance or to obtain [illegible] modified products. Recent [illegible] of composition, [illegible] special pavement pretreatments; accelerated laboratory tests to predict durablity; evaluation of the nature or cause of field failure; film thickness; and developments in reflectorizing glass beads. Each of these is discussed separately.

Composition

During the period from about 1940 to 1960, considerable research effort was directed at the pigment portion of traffic paint. In recent years, however, this aspect has received little attention from the State highway departments. Recent research and development effort on paint composition largely has been directed to the organic binder, commonly referred to by paint specialists as the nonvolatile vehicle. The most commonly used organic binder in conventional traffic paints purchased by State highway departments is the drying oil modified alkyd resin (*1*). Other vehicles used by these agencies in order of their decreasing prominence are: chlorinated rubber-alkyd, dispersion resins, phenolics, chlorinated rubber-petroleum hydrocarbon, and polyvinyl toluene polymer. A 1968 report by the Georgia Institute of Technology included results of laboratory and field tests on a wide variety of traffic paint formulations (*5*). The authors of that report concluded that oil modified alkyd, as well as chlorinated rubber-alkyd traffic paint, exhibited good performance—much better than most of the other types evaluated. In a separate report, it was shown by the New York Department of Transportation that, compared to a number of other conventional solvent-based paints, oil-modified alkyd paint gave the best performance (*2*). The New York Department of Transportation also conducted field tests and found that latex or emulsion-type paints did not compare favorably with their standard modified alkyd paint (*6*). The work in New York indicated that the latex paints had a useful life of only three or four months on either concrete or bituminous pavement. The latex paints performed poorly regardless of whether they were applied at ambient temperature or preheated and applied hot. Latex paints were also found to give poor adhesion in field tests reportedly conducted by the New Jersey State Department of Transportation (*7*).

Federally-aided research at North Dakota State University is currently being conducted on traffic paint vehicles that have the potential of reacting with the pavement substrate to provide good bond and durability. In this research, isocyanate-based paints are being studied for reaction with portland cement concrete pavement. At present, no significant trends have been reported. Current research at the Central Office of Public Roads in Poland is directed toward the development of polystyrene paints, but no conclusions are yet available.[3]

Special pavement pretreatments

In a 1957 study by Keese and Horn, it was reported that, compared to simple brooming or use of compressed air alone, sandblasting a portland cement concrete pavement to clean the surface prior to striping increased paint life substantially (*8*). In that study, sandblasting costs were reported to be 10 cents per linear foot. Regular cutting sand with an airflow of 210 cubic feet per minute and 110 p.s.i. compression was used for blasting. In a report issued by the New York Department of Transportation in 1969 (*2*), it was determined that neither acid etching nor synthetic rubber primer increased the durability of traffic paint on either concrete or bituminous surfaces, regardless of the ages of the surfaces. It was also reported that preheating the paint and applying it hot did not increase durability. According to a 1966 study by the Corps of Engineers, adherence of paint to portland cement concrete pavement can be improved by pretreating the pavement with a 50-50 mixture of boiled linseed oil and mineral spirits (*9*). It was shown in their field tests that, compared to an untreated test section, adhesion to the pretreated pavement was superior. This finding should be of value to northern States where concrete bridge decks are pretreated with linseed oil to prevent damage by deicing salt.

Laboratory tests and cause of traffic paint failure

For a great many years an urgent need has existed for an accelerated laboratory wear test that would accurately predict field durability and properly rank traffic paints according to their field performances. Over the past several decades, much research has been directed to developing such laboratory-wear or abrasion tests. Simulated traffic wheels, Taber abrasion tests, falling sand tests, and other methods have been tried, but none has gained universal acceptance because of poor correlation with field results. A recent additional effort by the Georgia Institute of Technology to develop a laboratory wear test that would correlate well with field performance (*5*) was unsuccessful. During 1968, research reportedly was underway at the Federal Institute of Road Research in Germany to develop an accelerated laboratory durability test.[4]

Apparently, some traffic paint specifications that are based solely on physical tests are unable to adequately assure good paint performance on the road. This fact was confirmed by a 1964 report by the U.S. Naval Civil Engineering Laboratory (NCEL) in connection with Federal Specification TT-P-85b (*10*), which is for reflectorized traffic paint for airfield runway markings. The NCEL report stated that mere conformity with the requirements of Federal Specification TT-P-85b did not guarantee a paint with good field service, nor did the test results obtained on a wide variety of conforming paints correlate well with field service. In particular, NCEL reported that the laboratory bleeding test was of little value in predicting field bleeding behavior. Specification TT-P-85b also stipulates a falling sand abrasion test to assure good durability, but the merits of this test are rather questionable on the basis of the NCEL findings.

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[3] Highway Research Information Service, Highway Research Board, Washington, D.C.

[4] Highway Research Information Service, Highway Research Board, Washington, D.C.

Perhaps the major reason that abrasion-type laboratory tests have never become fully reliable or firmly established is that actual field failure involves a different mechanism. According to recent research results from an ongoing study by the State Highway Commission of Kansas, chipping or loss of adhesion, rather than abrasion loss, is a major cause of traffic paint failure on portland cement concrete. This observation, particularly with respect to concrete surfaces, was also noted by the Texas Highway Department and was reported in the literature by others some years ago (*11*). Work currently underway in Colorado confirms that traffic-paint chipping is a prominent cause of failure (*12*).

Paint film thickness

The most predominant wet film thickness of traffic paint used by most State highway departments is 15 mils (*1*). Use of this thickness may have become a general practice as a combined result of field experience and published research results. More than 10 years ago, the Michigan Department of State Highways, investigating the field performance of various wet film thicknesses from 12 to 21 mils (*13*), reported that the thicker films were more durable but that the additional life of paint thicker than 16 mils was not directly proportional to the additional thickness used. Accordingly, it was concluded that to increase the wet film thickness beyond 16 mils was not economical. Thus, the general practice of applying a wet film of 15 or 16 mils seems to have some substance. This practice is keyed to the use of conventionally graded glass beads.

In more recent work reported in 1965, researchers at the Georgia Institute of Technology reported that paint durability, and even night visibility, was improved as wet film thicknesses within the range of 10 to 20 mils (*5*) were increased. They specifically noted that wet film thicknesses of 10 mils were generally incapable of properly binding drop-on beading, presumably of conventional gradation. Their field work included a two-coat application of 15 mils each, which provided superior performance. However, it was postulated in the report that, for single-coat applications, 20 mils would seem to be the practical upper limit for the types of paints investigated. The Kentucky Department of Highways, experimenting with multiple film applications of 15 mils each, reported informally that multiple coats provided improved durability over a single 15-mil thickness, but that the comparative economics had not been fully established.

On the other hand, some of the research now being pursued seems to contradict part of these findings. For example, researchers at North Dakota State University, conducting a traffic paint study supported financially by the North Dakota State Highway Department and the Bureau of Public Roads, have stated that wet films much thinner than 15 or 16 mils are required for good paint adhesion and subsequent durability. This contention is based on limited field tests and on the premise that traffic paint fails more often through lack of adhesion than through abrasion. However, only limited field evidence has yet been presented to support this hypothesis. In an interim report on a study by the State Highway Commission of Kansas it was stated that both reflectivity and durability were increased when paint film thickness was reduced from 15 to 10 mils (*14*). This work is being performed in conjunction with an investigation of reduced bead application rates.

Hence, at present, evidence as to the optimum film thickness seems to conflict. Perhaps test conditions and even the bead application rate are some reasons for the different findings. The work in Kansas on films thinner than 15 or 16 mils was conducted on previously striped areas where remnants of the old stripe still remained under the newly applied stripe. Consequently, the thicker the new stripe, the more the older paint remnant lost its adhesion to the pavement. Moreover, where larger accumulations of old paint were evident, it was determined that chipping losses increased. The Kansas work also included an investigation of reduced bead application rate, which will be more fully considered in the next section.

The optimum film thickness of traffic paint may therefore depend on whether striping is applied to a previously unstriped area or to an area on which remnants or accumulations of old paint still exist under the new striping. In addition, it is becoming apparent that optimum film thickness is closely related to bead gradation and bead application rate.

Reflectorized beads

Small glass beads not only provide night visibility for traffic stripes but are reported to contribute as well to the overall durability of the paint binder (*5*). Within the last several years, considerable attention has been given to the role of traffic paint beads. A recent article by J. M. Dale, based on theoretical considerations, raised many questions about the adequacy of current State highway specifications for bead gradation and application rate (*15*). The parameters of glass beads, including the best index of refraction, are controversial factors, which fortunately are under intensive investigation at present.

In most State highway departments, the present practice is to use beads of a standard refractive index—1.50+ as opposed to 1.65 or more—and an application rate of about 6 pounds of drop-on beads per gallon of paint (*1*). Generally, the bead-size gradations range from a maximum of 20 mesh to a minimum of about 100 mesh (*15*). These common practices will now be examined in the light of recent research and development findings.

Regarding refractive index, the New York Department of Transportation reported that although the special 1.65+ index bead, measured instrumentally, gave slightly better initial reflectivity, the improvement was hardly noticeable to the driver, and the measured quantitative difference diminished and finally disappeared as the stripe exposure time increased (*16*). It was concluded that use of the standard index bead, 1.50+, at a savings of 2 to 3 cents per pound, was preferable. The results in New York were applicable to both concrete and bituminous pavements. In a recent report by the Colorado Department of Highways, similar results were cited (*12*).

The application rate of drop-on glass beads to traffic paint is another parameter that is receiving current research attention. On the basis of an ongoing research study, being conducted in conjunction with the study of films thinner than 15 or 16 mils, the State Highway Commission of Kansas reports that the beading rate could be reduced from 6 pounds per gallon of paint possibly to 4 pounds per gallon with no loss in reflectivity or durability and with resultant economic savings (*14*). According to theoretical considerations and practical laboratory tests, an intermediate bead application rate provides better reflectivity than either a very high or very low rate (*17*). In a recent report by the Colorado Department of Highways it was also shown that an application rate of 4 pounds per gallon was both more economical and more effective than a rate of 6 pounds per gallon (*12*). However in that work, other special parameters, such as special gradation and specially coated beads to afford a flotation feature, were involved.

Bead gradation is also undergoing research consideration. The current gradation specifications used by the State highway departments differ considerably. Generally the sizes specified range from a top size of 20 mesh (100 percent passing) to approximately 100 mesh size (*15*). Highway engineers are interested in obtaining an optimum gradation and more uniformity between the gradation specifications of the State highway departments. In the previously cited article by Dale (*15*), it is suggested that fewer than 30 percent of the beads of current conventional gradation for drop-on applications are effective when they are first applied. It is claimed that many of the beads of the small size fraction are immediately submerged in the paint and therefore are not initially effective. It can be argued that these submerged beads never become effective if the paint fails by chipping. On the other hand, if the paint wears out mainly through abrasion, the initially submerged beads become effective later in the life of the film. Previously cited research supports the theory that adhesion or chipping failure is a significant factor in the life of the paint film.

Field test results, reported some years ago for special gradations of glass beads obtained from a single source (*18*), indicated that coarse beads, 20 to 40 mesh, initially gave much higher reflectance than either fine beads, 40 to 100 mesh, or conventionally graded beads, 20 to 100 mesh. However, this superiority was not maintained for a long period. After about six months exposure, the reflectivity of the coarse gradation dropped below that of the other two gradations, indicating that the coarse beads ultimately are removed from the binder by traffic action, even though [illegible] are initially more effective because a greater proportion is exposed at the surface. In recent studies it has been shown that coarse beads, 40 mesh or larger, are lost by the binder soon after installation because they are poorly anchored (*12, 14*). Conversely, the extremely fine beads, 80–100 mesh, sink into the paint and never become effective reflective units when the paint fails from chipping or lack of adhesion. Accordingly, without considering cost factors, the optimum gradation should be one that is between a maximum size of 40 mesh and a minimum size of about 80 mesh, provided that a conventional wet film thickness of 15 or 16 mils is used.

The Colorado field experiment cited earlier is now being conducted on uniformly graded beads of a narrower gradation range than conventional gradations (*12*). The beads contain a coating that gives them a flotation property that is claimed to make them float but be partially immersed in the paint film. Presumably this property allows more reflectorizing units to be available at the paint surface. In interim reports from the Colorado experiment, it is shown that the special beads applied at a rate of 4 pounds per gallon provide reflectance equal to or greater than that of the conventional beads normally used by the States, which are applied at a rate of 6 pounds per gallon. According to the Colorado report, use of the special beads at the reduced application rate provided significant cost advantages. After 10 months, field tests indicated no evidence of any serious reversals in reflectance or of poorer durability. These findings have been confirmed by the Nebraska Department of Roads which informally observed the performance of such beads. The Nebraska observers stated that the flotation feature seemed more important than closer grading, as beads of the same grading but without the flotation property did not provide the same high degree of reflectance.

One predominant factor for good reflectivity seems to have been established by the results of past and current research—a bead must be slightly more than one-half immersed in the paint binder to obtain maximum retroreflectivity at high incidence angles (*17*).

Rapid-Dry Traffic Markings

There has been a recent upsurge of interest in rapid-dry traffic markings among State highway departments. One reason for this interest is the desire to disrupt traffic flow as little as possible during striping operations on existing highways where traffic densities are increasing. Another reason is economics; the cost of placing and removing the traffic cones needed for conventional striping can be partly offset by using the more expensive quick-dry markings, which do not require protective cones.

[illegible] study was conducted on [illegible] energy to heat and [illegible] application [illegible] The method was adjusted [illegible] it was [illegible] different [illegible] energy [illegible] dissipation [illegible]

The use of plasticized sulfur, which is melted in a flame or kettle and hot-sprayed onto the pavement, is being investigated as a quick-set marking by the Phillips Petroleum Company. Another material that seems somewhat similar, called *Sullarzan*, was reported in the British literature (*20*), in which the following properties were claimed: It is obtained by polymerizing sulfur (it may, in fact, be a mixture of polysulfide polymer and unpolymerized sulfur); it is available in solid granular form; it was developed by the Societé Nationale des Petroles d'Aquitaine and is being introduced for trial in France and England; it is available in any color, presumably is applied in molten form, and dries, or sets, to a no-pick-up condition in 15 seconds; it contains glass beads and may even be applied to *wet* or *icy* roads (the latter claim is rather difficult to believe) and its covering rate is 400 grams per square meter. Its producer claims high durability—"6 times the durability of existing road marking materials." No evidence of rigorous and conclusive field research on the merits of sulfur-based markings by highway agencies is known to the author.

Several fast-dry, solvent-based paints were introduced in the last decade, one of which is the *Nite-Line* paint produced by Prismo Universal, Inc. Claimed to dry in 3 minutes or less, the paint is heated to 165°F and is applied by spray from special equipment. Glass beads are blown by compressed air into the wet binder. Application speed is said to be 8 m.p.h., and no traffic cones are used when the material is applied to the pavement. A follow-up truck about 500 yards behind the application truck temporarily keeps traffic off the wet stripe until it dries. The cost of the material, including glass beads, is about $6 per gallon. Reportedly, this product has been used in New York City and Baltimore, as well as in Pennsylvania, Florida, and Arkansas. In a recent article, it was reported that the Michigan Department of State Highways used a heated fast-dry paint, which may be similar to the above-mentioned product (*21*), except that the dry time claimed in the Michigan application was 1½ minutes rather than 3 minutes.

A similar fast-dry paint product, called *Green-Lite Liquid Striping Compound*, is being sold by the 3M Company. This solvent-based paint has a high solids content and is heated to 250°F and applied by airless spray. Dry time is claimed to be only 2–20 seconds, or an average of 10 seconds. The striping equipment can be purchased. The material contains pre-mixed beads and is applied at a wet-film thickness of 15 mils, which provides a dried film of about 11 mils. The producer claims that the material reacts with the air immediately when it is sprayed and that this aids in speeding up the dry time. Again, no protective cones are required for application. The durability of the paint is said to be equal to that of conventional striping, and material costs are about $8 per gallon. The maximum speed of the striper is 10 m.p.h. The company states that this material is being tried out in Florida, Michigan, Missouri, California, and Colorado.

Similar heated, fast-dry proprietary paints, such as *Hot Line* produced by the Baltimore Paint and Chemical Corp., are also available from other producers. Within the last year or two, the Texas Highway Department has been experimenting with a fast-dry, heated paint based on chlorinated rubber, chlorinated paraffin, and alkyd resin. The paint has a low solvent content and requires only moderate heating. The needed heat can be obtained from the liquid cooling system of the engine on the striping truck, thereby eliminating the need for special auxiliary heaters like those used for some of the other proprietary fast-dry, heated paints. This material is applied by airless spray and reportedly dries in 2 minutes when heated to about 120°F, or alternatively dries in 4 or 5 minutes when applied without heating. Material cost of this paint is estimated to be about $2.30 per gallon. This material may be fairly similar to the British product called *Clearlane*.

Several other types of proprietary products have a rapid, almost instant, setting property. These include the hot-extruded thermoplastic striping and hot-sprayed plastic materials that are applied as rather thick coatings. However, because such materials were primarily developed to provide long-lasting stripes, they will be discussed more fully in the following section.

Semi-Permanent Markings

Materials that have been developed to serve as semi-permanent markings include preformed glue-down plastic stripes, hot-extruded and hot-sprayed thermoplastic stripes, and plastic or ceramic raised button-type markers. Compared to conventional stripes, they were developed mainly to provide long-range economic benefits, although the raised button-type markers are also intended to provide improved visibility in rain or fog.

Preformed glue-down plastic stripes contain an adhesive backing and are mainly used on bituminous pavement in high-density urban areas—usually for crosswalks and stop lines. Their use on open highways has been rather limited. A report by the Arkansas State Highway Department indicated that such materials do not provide adequate durability, mainly because their adhesion to concrete surfaces is poor (*22*). Somewhat better performance was reported when the material was applied to bituminous surfaces. Installed costs on a contract basis were stated to be 55 cents per linear foot of 4-inch-wide stripe. A research report by the New York Department of Transportation similarly indicated poor durability on open highways because the material was quickly dislodged and destroyed by traffic and snowplows (*23*). Proprietary products of this type produced in this country include *Plastix* and *Neff-Slabs.* An apparently similar material, called *Verynl,* was investigated by the New Zealand Transport Department in 1968. This material is claimed to have good wear and to be more economical than paint for pedestrian cross-walks. *Verynl* contains polyvinyl chloride, is 2.5 mm. thick, and is applied with standard mastic. It was reportedly tried out in this country in Los Angeles.

There has been considerable research in this country on hot-extruded thermoplastic stripes, which are applied at a thickness of about one-eighth inch. Such materials are sold under the trade names of *Permaline* and *Catatherm*. A comprehensive survey and report on these materials, prepared this year by the Bureau of Public Roads (*24*), was based on the results of a large number of installations throughout the United States. The major conclusion and recommendations in that study were as follows:

- Hot-extruded thermoplastic materials were much more durable on bituminous pavement than on concrete pavement. The material is more durable on older concrete than on new concrete.
- Snowplow activity dislodged the material from the pavement, particularly on concrete.
- A factor limiting the economic value of thermoplastic striping on bituminous pavement is the limited maintenance-free life of the bituminous surface.
- Unremoved layers of old paint adversely affect the adhesion of thermoplastic striping to the pavement.
- Under conditions of little or no snowplow activity, thermoplastics were found to be more economical than paint striping, provided the traffic density was appreciable. For bituminous pavements, the daily density should exceed 6,000 vehicles per lane; for concrete pavements, the density should exceed 9,000 vehicles per lane. Under conditions of moderate snowplow activity, the use of thermoplastics could be justified, provided that traffic density levels are high enough. Finally, under conditions of severe snowplow activity, there is little economic justification for the use of hot-extruded thermoplastics.
- Guidelines in the report enable the selection of the most economical material—paint or thermoplastic—for different traffic densities and degrees of snowfall.

An interesting sidelight to investigations of thermoplastic striping materials is a 1965 report by California (*25*). Contrary to some beliefs, it was shown that thermoplastic striping presented no substantially greater skidding hazards than conventional striping.

Thick coatings of hot-sprayed plastic material for semi-permanent markings is a comparatively new development, exemplified by the proprietary product called *Hot Spray Plastix*. This hot-sprayed material, applied at a thickness of 90 mils, costs approximately 21 to 26 cents per linear foot for 4-inch-wide stripes and hardens in seconds. At present, experience has been more limited with this material than with hot-extruded thermoplastics. The product is being tried in Philadelphia, on the Penn-Lincoln Parkway near Pittsburgh, on I-495 near Washington, D.C., and on various expressways in and around New York City.

Raised button-type markers are also a form of semi-permanent marking, but because they also serve to improve night-wet visibility, they are discussed in the following section.

Markings for Improved Night-Wet Visibility

Raised button-type markers, used widely in some of the far western States, notably Washington and California, actually serve two purposes: they are long-lived and therefore economical, and they provide excellent night-wet visibility. Their use has been limited largely to areas that are free from snow and snowplow activity. Raised markers are used extensively in the snow-free areas of California (*26*). Patterns of four white nonreflective markers placed 3 feet apart are separated by 15-foot gaps. One reflective-type marker is placed in each gap on curves and in alternate gaps on tangents. According to the California Department of Public Works, the best reflective marker in use is an acrylic or ABS plastic-encased corner cube reflector. The Department's experience indicates that the raised marker is much more durable than conventional paint. The Washington Department of Highways also has reported good experience with raised markers in snow-free areas (*27*). Raised markers were reported to be superior to standard traffic paint in terms of durability, driver preference, and night-wet visibility, and their cost for a 10-year period was claimed to be compatible with the cost of standard paint striping. The annual cost of marker maintenance was one-fourth that of paint-striping maintenance. The California Department of Public Works recently reported on the development of a rapid-set epoxy adhesive for raised markers (*28*), which is based on the use of a polymercaptan curing agent that cures epoxies in thin films at a rate that is 7 to 10 times faster than the conventional curing rate of epoxies. The new adhesive can be used at temperatures lower than 30° F.

Raised markers perform poorly in snowy areas that are subjected to snowplow activity, as proved by experiments in Arkansas (*22*) and California (*29*), in which raised buttons failed badly. In California, failure occurred even when precast markers were inset in the pavement to obtain a lower profile. Poor performance in snow areas was also evident when epoxies were cast and cured-in-place in cut-out pavement sections that were finally finished off flush with the pavement surface.

When applied in snow-free areas, raised button-type reflective markings have substantially solved the problem of good night visibility under wet weather conditions. Present emphasis in research is to extend this solution to the snow-belt area of the country and also, if possible, to develop more economical and durable systems. In an interesting research study just completed in the State of Washington, regular raised markers were used in a snow-belt area, and snowplow blades were equipped with neoprene rubber bits to prevent marker damage. During two winter seasons at three separate test sites, the synthetic rubber bits were effective in removing snow except when the temperatures consistently remained below freezing. When semi-thawing conditions existed or could be induced by deicing chemicals, the rubber-tipped plows were very effective. Although initial costs were higher, the rubber bits did not wear out as quickly as steel bits and actually cost less per mile plowed. The most interesting part of this experiment was that few raised markers were lost at the test sites. At one site with concrete pavement, marker loss attributed to the rubber-tipped plow blades was from 1 to 2 percent during one winter season. At another site with bituminous pavement, marker losses were from 3 to 5 percent for a single winter. At the third site, no definite count was made, but the State reported from informal observations that little marker loss occurred in one winter.

Another development to obtain night-wet visibility in northern States where snowplow operations are extensive is the so-called *snow-plowable*, raised, reflective marker under investigation in both Pennsylvania and Texas. The reflective element is a high strength acrylic plastic that is based on corner-cube retroreflection and is contained in a steel casting with two keels. The keels are cemented with epoxy resin into grooves cut into the pavement. The casting also contains protective runners that direct snowplow blades over the marker, presumably without damage. The marker is installed in the skip zone between dashed stripes, generally in every other skip zone on tangents and each skip zone on significant grades and on horizontal curves. The estimated installed cost of each marker is approximately $4, and the replacement cost of each reflective element is estimated at 25 cents. The Texas project was recently initiated but the Pennsylvania installation on the Schuylkill Expressway in Philadelphia has been in progress since 1967. Informal reports on the latter investigation indicate that although very little snowplow damage occurs to the steel castings, the plastic reflectors have been badly damaged.

Other ideas for more economical, snowplow-resistant systems for night-wet visibility are being investigated in an ongoing study at the Georgia Institute of Technology that is sponsored jointly by the State Highway Department of Georgia and the Bureau of Public Roads. Researchers are looking into the possibilities of a corrugated or textured hot thermoplastic stripe containing beads. The thought behind this work is that the peaks of the corrugated thermoplastic stripe will not be immersed in water during heavy rains and, thereby, will provide retroflection in wet weather. The study also includes use of reflectorized aggregate chips in a suitable binder to obtain the same results. Two test sites, one snow-free and the other subject to snowplow activity, will be used in the investigation. Performance will be compared with conventional raised markers.

Another approach to the night-wet visibility problem is under study in the National Cooperative Highway Reaserch Program managed by the Highway Research Board. This study, NCHRP Project 5-5 entitled *Nighttime Use of Highway Pavement Delineation Materials*, is in progress at the Southwest Research Institute. It deals with the use of quarter-inch-diameter glass beads that are dropped into a cast-in-place epoxy marker, which is then allowed to set and harden. The large-bead concept was developed in a preliminary phase of the same study (*30*). The researchers had developed the concept that one-quarter-inch diameter beads half-imbedded in an appropriate binder will provide a profile that will not be completely inundated by a film of rain water and, thereby, will provide adequate night-wet visibility. The researchers believed that this concept would be less expensive than commercial raised markers and that the lower overall profile would be subject to less damage by snowplows. In the current phase of the study, a road machine that casts the epoxy, deposits the beads, and then moves on to the next marker location was developed. Installations of this type have been placed in Texas, California, New Jersey, North Carolina, and Florida. The study is to be completed shortly and the final report is under review.

In another novel experiment to obtain improved night-wet visibility now being conducted by Utah State Department of Highways (*31*), small longitudinal grooves are cut into concrete pavement to an overall width of 4 inches. At intervals, transverse grooves are cut across the longitudinal grooves to provide positive rain water drainage away from the longitudinal grooves to the low side of the crowned pavement surface. The longitudinal grooves generally are a quarter-inch deep. The grooving is then striped with conventional reflectorized traffic paint. Theoretically, the grooved striping will be more durable than conventional striping, as it is recessed and not subjected to traffic and snowplow abrasion. More importantly, the stripe should be more visible in rainy weather than conventional striping because rain water is drained away from the reflectorized grooved surfaces. The study is still in progress and no data are yet available on the relative economics of such striping, although recent indications are that the life of the grooved striping is better than that of conventional striping. One important development so far is that the grooved striping is more visible than standard striping during a rainstorm—either at night or during the day. The study will include several different grooving designs to determine the one that gives optimum performance. In addition, the State highway department has suggested that grooves be formed in a newly placed pavement—either fresh concrete or hot bituminous pavement—and has offered appropriate plans. There is considerable interest in this experiment from the standpoint of a snowplow-resistant marking that can provide night-wet visibility. As a result, at least two other States, New York and California, plan to investigate this approach.

Temporary Lane Markings

Temporary lane markings for construction detours are of interest to many State highway departments. In 1961, a road machine used by the Missouri State Highway Commission to remove temporary stripes was described in the literature (*32*). Containing a 9-horsepower engine equipped with 92 steel cutters, the machine cost $758. It could operate at speeds as high as 10 feet per minute. The cost of each steel cutter was 20 cents and each could last as long as four working hours.

More recently, in 1965, the Michigan Department of State Highways reported an evaluation of temporary removable lane marking tape (*33*). It was claimed that grinding equipment to remove temporary paint was not fully effective and that paint residues remained to confuse the driver. The evaluated tape is 20 mils thick and is available in 4- to 6-inch-wide rolls. The core is soft aluminum and the top is a white or yellow beaded vinyl material. The backing contains a pressure-sensitive asphaltic adhesive and is used in conjunction with a pavement primer to promote good adhesion. It is recommended for use as a temporary marking instead of conventional paint, which is both difficult and costly to remove. The Massachusetts Department of Public Works, as a result of its own study completed in 1968, also recommended use of removable tape for temporary lane markings. The tape was compared with a water-based temporary paint and determined to be much more preferable.

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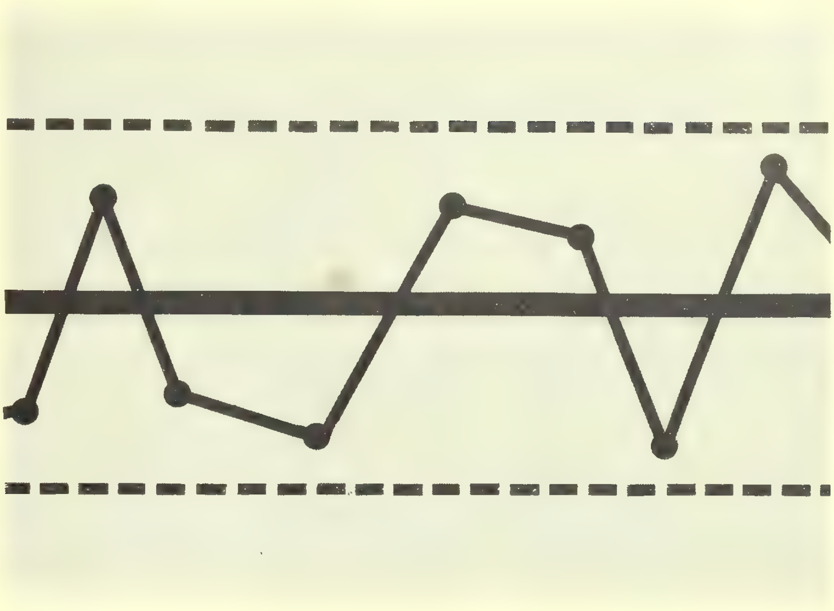
(*Continued on p. 264*)

Quality Assurance in Highway Construction

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Part 6—
Control Charts



**Reported by EDWIN C. GRANLEY,
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This is the final part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, Part 2.—Quality Assurance of Embankments and Base Courses, Part 3.—Quality Assurance of Portland Cement Concrete, Part 4.—Variations of Bituminous Construction, and Part 5.—Summary of Research for Quality Assurance of Aggregate were presented in previous issues of PUBLIC ROADS.

Introduction

TEST MEASUREMENTS portrayed graphically demonstrate the familiar adage, *a picture is worth a thousand words.* One type of graphical portrayal, the control chart, is used extensively by industry in quality-assurance procedures. Many of the industrial control-chart principles are just as applicable to process control and acceptance inspection in highway construction as they are to industrial techniques.

Control charts show cumulative trends in dimensional or physical properties within maximum and minimum limits that denote acceptable production. They not only indicate when established limits have been exceeded, but also provide the means to anticipate and correct causes that tend to promote the production of defective products. Thus, the use of control charts is an application of the principle that an ounce of prevention is worth a pound of cure.

In industrial statistical quality control methods, certain variations in product quality are classed as *chance* variations. Such variations obey the laws of probability as inevitably as the flipping of a coin. Little can be done to change these variations except to revise the control process.

In addition to these chance variations other variations in quality, systematic variations, can be attributed to *assignable causes,* such as differences among equipment, workers, materials, time, etc., and the interrelations of each to the others. Knowledge of the behavior of chance variations is the basis of control-chart analysis. If data vary in a pattern that conforms to the applicable statistical distribution, an assumption is made that no assignable causes exist. These data are said to be *in control.* If the data do not follow the statistical distribution, it is considered that assignable causes are at work and that the process is *out of control.* When the process is *in control,* the distribution of variations for large numbers of items are predictable.

According to Duncan (*1*), [1] ". . . a control chart is a device for describing in concrete terms what a state of statistical control is; second, a device for attaining control; and, third, a device for judging whether control has been attained."

The construction and use of a control chart can be explained simply. Samples of a given size are taken randomly from a process at more or less regular intervals and tested. If no assignable causes exist, these test values will be distributed in a definite pattern. If the distribution is normal, the pattern will assume the shape shown by the normal curve in figure 1. These data from the tests on the samples can be used to construct the control chart shown in figure 2. Actually, the normal curve has been tipped on its side and the values plotted in the horizontal plane on a time or unit of production basis. If a significant number of average ($\bar{X}$) values are plotted and all of them fall within the control limits and are randomly distributed about the average, that is, show no trends, then it can be said that the process is in a state of statistical control. If the data do not conform to this pattern, but show cycles or runs above or below average or outside the limit lines, the process should be investigated and the assignable causes ascertained. If all points fall within the control limits and are randomly distributed, assignable causes are not necessarily absent. However, the conclusion that chance causes alone are at work is acceptable, and a search for assignable causes would be unprofitable.

Types of Control Charts

Control charts now in use fall into two categories: Charts of attributes and charts of variables. Usually, control charts of attributes are associated with the *go* or *no go* discipline. They show one of two types of information: (1) The fraction defective or percent of items conforming or nonconforming to specifications, or (2) the number of defective items. Although the attributes may often be measurements, mostly they are visually inspected properties like finish, scratches, missing parts, cracks, or functional ability. On the other hand, control charts of variables are used for values such as p.s.i. density, thickness, length or other such units reported from testing.

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[1] Italic numbers in parentheses identify the references listed on page 260.

Whether to use plans for attributes or variables to control a process in highway construction cannot be answered here. The use of either will be dictated by the process or product to be controlled, and each will provide an effective tool to guide the State and the contractor. The only requisites are that the process be continuous, that the chart be visibly displayed, and that the charts be available to everyone concerned and correctly interpreted. Only control charts for variables will be considered in the remainder of this article.

Control charts can be constructed from test data in the form of individual measurements, averages, standard deviations, and ranges, either singly or in combinations. Again, the form to use will be dictated by the process to be controlled and the economics of the project.

Variables charts fall into two categories: (1) standards given and (2) standards unknown. The usual way to develop a control chart for the standards-given category is to analyze past data and use the information obtained to set up the charts for present and future production. Analysis of the past data will provide values of the average, $\bar{X}$,′ the range R,′ and the standard deviation σ, of the population for calculating the midpoint and limit-line positions on the chart. Usually, the charts are used in pairs—$\bar{X}$ and R or $\bar{X}$ and σ.

Computation of Limit Lines

To determine the respective limit lines for a normal population, when standards are known, the following formulas (*2*) are used:

$\bar{X}$ Chart:

$$\begin{aligned}\text{Center line} &= \text{℄} = \bar{X}'\\ \text{Upper control limit} &= \text{UCL}\\ &= X' + A\sigma'_x\\ \text{Lower control limit} &= \text{LCL}\\ &= \bar{X}' - A\sigma'_x\end{aligned}$$

R Chart:

$$\begin{aligned}\text{℄} &= d_2\sigma'_x\\ \text{UCL} &= D_2\sigma'_x\\ \text{LCL} &= D_1\sigma'_x\end{aligned}$$

σ Chart:

$$\begin{aligned}\text{℄} &= C_2\sigma'_x\\ \text{UCL} &= B_2\sigma'_x\\ \text{LCL} &= B_1\sigma'_x\end{aligned}$$

Where the constants A, B_1, B_2, D_1, D_2, C_2, and d_2 for different sample sizes are given in tables of control chart constants like table 1.

Sometimes control charts must be constructed when historical data are not available for analysis and, consequently, the parameters (standards) are unknown. If the standards are unknown, data from 20 to 25 samples, consisting of four to five measurements each, are needed to compute the limits for the charts. Averages, ranges, or standard deviations of measurements on each sample are averaged to give $\bar{\bar{X}}$, $\bar{R}$ or $\bar{\sigma}$, respectively. Again, the charts are used in pairs—$\bar{X}$ and R or $\bar{X}$ and σ. The following formulas can be used to compute the center lines and the control limits for the charts:

If $\bar{\text{X}}$ *and* R *charts are used:*

$\bar{X}$ Chart:

$$\begin{aligned}\text{℄} &= \bar{\bar{X}}\\ \text{UCL} &= \bar{\bar{X}} + A_2\bar{R}\\ \text{LCL} &= \bar{\bar{X}} - A_2\bar{R}\end{aligned}$$

R Chart:

$$\begin{aligned}\text{℄} &= \bar{R}\\ \text{UCL} &= D_4\bar{R}\\ \text{LCL} &= D_3\bar{R}\end{aligned}$$

If $\bar{\text{X}}$ *and* σ *charts are used:*

$\bar{X}$ Charts:

$$\begin{aligned}\text{℄} &= \bar{\bar{X}}\\ \text{UCL} &= \bar{\bar{X}} + A_1\bar{\sigma}\\ \text{LCL} &= \bar{\bar{X}} - A_1\bar{\sigma}\end{aligned}$$

σ Charts:

$$\begin{aligned}\text{℄} &= \bar{\sigma}\\ \text{UCL} &= B_1\bar{\sigma}\\ \text{LCL} &= B_3\bar{\sigma}\end{aligned}$$

The values of constants A_1, A_2, B_3, B_4, D_3, and D_4 are listed in table 2.

Limits established by use of the factors from tables 1 and 2 are based on statistical probability and will satisfy the statistician but not necessarily the engineer. Usually the engineer wants the product to meet specification requirements that are based on engineering considerations, and he may not care whether the process is in statistical control. Many engineers prefer that the specification be used to develop the control chart. This can be done, for example, on a two-way specification by using the target as the centerline and the tolerances as the upper and lower limits. If the test results show nonconformance, either the process needs to be changed, sampling and testing techniques altered, processing improved, or the specifications changed to meet existing conditions. The engineer is thus faced with two alternatives: either change the specifications or insist that the requirements be met.

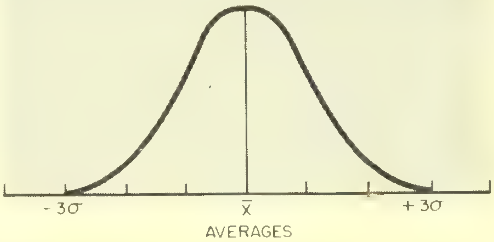

Figure 1.—Distribution of chance variations in a test sample measure of quality.

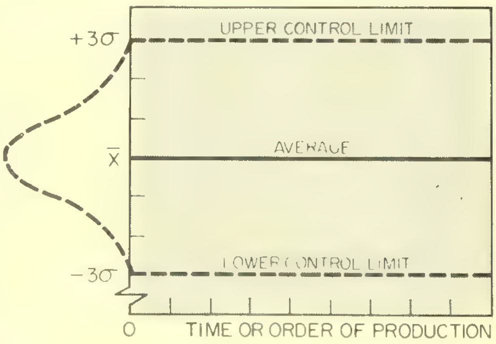

Figure 2.—Theoretical basis for a control chart.

How the Chart Works

The average, $\bar{X}$, chart shows shift of process average. The range, R, or standard deviation, σ, charts show process dispersion or spread. Usually, the range, R, chart rather than the standard deviation, σ, chart is used to measure subgroup dispersion. Although both present similar trends, the range values for the R chart are much easier to compute and explain. Charts of individual test values can be used to depict both the shift of average and the dispersion, though not as efficiently as the combination of average and range charts.

Following are a few of the many warning signals that have been developed to indicate out-of-control processes (*3*):

Individual charts

- 1 test value greater than 2.33 standard deviation from centerline.
- 3 consecutive test values greater than one standard deviation above or below the centerline.
- 11 consecutive test values on same side of centerline job mix target.

Average and range charts

- 1 average or range value outside control limit lines on R and $\bar{X}$ chart.
- 2 consecutive averages outside 2 σ limits above or below the centerline.
- 7 consecutive values on either side of centerline for $\bar{X}$ charts or above centerline for R charts.

These signals are based on statistical probabilities; however, a 3- or 4-point consecutive trend toward either limit or above or below the centerline should be considered a warning. Diagnosis of the reason for the process being out of statistical control is an engineering consideration. The chart can only point out that something is wrong.

Examples of Use

At present, several States are experimentally using variables control charts on several items of construction.

In 1967, the South Carolina State Highway Department (*4*) used specification-based control charts to control and accept production of bituminous hot mix on an Interstate job. A chart on which extraction test results were plotted as individual measurements, averages

Table 1.—Factors for computing control chart lines—standards given

Number of observations in sample, n	Chart for averages	Chart for standard deviations			Chart for ranges		
	Factors for control limits	Factors for central line	Factors for control limits		Factors for central line	Factors for control limits	
	A	c_2	B_1	B_2	d_2	D_1	D_2
2	2.121	0.5642	0.000	1.843	1.128	0.000	3.686
3	1.732	0.7236	0.000	1.858	1.693	0.000	4.358
4	1.500	0.7979	0.000	1.808	2.059	0.000	4.698
5	1.342	0.8407	0.000	1.756	2.326	0.000	4.918
6	1.225	0.8686	0.026	1.711	2.534	0.000	5.078
7	1.134	0.8882	0.105	1.672	2.704	0.205	5.203
8	1.061	0.9027	0.167	1.638	2.847	0.387	5.307
9	1.000	0.9139	0.219	1.609	2.970	0.546	5.394
10	0.949	0.9227	0.262	1.584	3.078	0.687	5.469
11	0.905	0.9300	0.299	1.561	3.173	0.812	5.534
12	0.866	0.9359	0.331	1.541	3.258	0.924	5.592
13	0.832	0.9410	0.359	1.523	3.336	1.026	5.646
14	0.802	0.9453	0.384	1.507	3.407	1.121	5.693
15	0.775	0.9490	0.406	1.492	3.472	1.207	5.737
16	0.750	0.9523	0.427	1.478	3.532	1.285	5.779
17	0.728	0.9551	0.445	1.465	3.588	1.359	5.817
18	0.707	0.9576	0.461	1.454	3.640	1.426	5.854
19	0.688	0.9599	0.477	1.443	3.689	1.490	5.888
20	0.671	0.9619	0.491	1.433	3.735	1.548	5.922
21	0.655	0.9638	0.504	1.424	3.778	1.606	5.950
22	9.640	0.9655	0.516	1.415	3.819	1.659	5.979
23	0.626	0.9670	0.527	1.407	3.858	1.710	6.006
24	0.612	0.9684	0.538	1.399	3.895	1.759	6.031
25	0.600	0.9696	0.548	1.392	3.931	1.804	6.058
Over 25	$\frac{3}{\sqrt{n}}$	-----------	(1)	(2)	----------	------	-----

[1] $1-\frac{3}{\sqrt{2n}}$.

[2] $1+\frac{3}{\sqrt{2n}}$.

Table 2.—Factors for computing control chart lines—standards unknown

Number of observations in sample, n	Chart for averages		Chart for standard deviations			Chart for ranges		
	Factors for control limits		Factor for central line	Factors for control limits		Factor for central line	Factors for control limits	
	A_1	A_2	c_2	B_3	B_4	d_2	D_3	D_4
2	3.760	1.880	0.5642	0.000	3.267	1.128	0.000	3.267
3	2.394	1.023	0.7236	0.000	2.568	1.693	0.000	2.575
4	1.880	0.729	0.7979	0.000	2.266	2.059	0.000	2.282
5	1.596	0.577	0.8407	0.000	2.089	2.326	0.000	2.115
6	1.410	0.483	0.8686	0.030	1.970	2.534	0.000	2.004
7	1.277	0.419	0.8882	0.118	1.882	2.704	0.076	1.924
8	1.175	0.373	0.9027	0.185	1.815	2.847	0.136	1.864
9	1.094	0.337	0.9139	0.239	1.761	2.970	0.184	1.816
10	1.028	0.308	0.9227	0.284	1.716	3.078	0.223	1.777
11	0.973	0.285	0.9300	0.321	1.679	3.173	0.256	1.744
12	0.925	0.266	0.9359	0.354	1.646	3.258	0.284	1.716
13	0.884	0.249	0.9410	0.382	1.618	3.336	0.308	1.692
14	0.848	0.235	0.9453	0.406	1.594	3.407	0.329	1.671
15	0.816	0.223	0.9490	0.428	1.572	3.472	0.348	1.652
16	0.788	0.212	0.9523	0.448	1.552	3.532	0.364	1.636
17	0.762	0.203	0.9551	0.466	1.534	3.588	0.379	1.621
18	0.738	0.194	0.9576	0.482	1.518	3.640	0.392	1.608
19	0.717	0.187	0.9599	0.497	1.503	3.689	0.404	1.596
20	0.697	0.180	0.9619	0.510	1.490	3.735	0.414	1.586
21	0.679	0.173	0.9638	0.523	1.477	3.778	0.425	1.575
22	0.662	0.167	0.9655	0.534	1.466	3.819	0.434	1.566
23	0.647	0.162	0.9670	0.545	1.455	3.858	0.443	1.557
24	0.632	0.157	0.9684	0.555	1.445	3.895	0.452	1.548
25	0.619	0.153	0.9696	0.565	1.435	3.931	0.459	1.541
Over 25	$\frac{3}{\sqrt{n}}$	------	------------	(1)	(2)	------------	------	-----

[1] $1-\frac{3}{\sqrt{2n}}$.

[2] $1+\frac{3}{\sqrt{2n}}$.

of 5 per lot, and range of 5 per lot, was used. Charts were also prepared for bitumen content and each of the following sieves: ¾ in., ½ in., ⅜ in., No. 4, No. 8, No. 30, No. 100, and No. 200 for each 2,000-ton lot. A facsimile of a control chart used on the South Carolina project for controlling percent passing the No. 30 sieve is shown in figure 3. Similar charts were used both for the remaining sieves and for bitumen content. Control limit lines for individual measurements, averages, and ranges were superimposed on each chart. Limits for average and range were computed from standards known, using appropriate values from table 1 multiplied by a factor of (2.33/3.00) times the standard deviation to reduce them to 98 percent probability intervals. Limits on the chart for the No. 30 sieve were developed from a standard deviation of 2.5 percent, which had been determined in previous research. The limits were computed as follows:

Individual measurements at 98-percent level

Job mix tolerance = (2.33) × (2.5) = 5.8

$\therefore$

UCL = 38.0 + 5.8 or 43.8 percent

LCL = 38.0 − 5.8 or 32.3 percent

Averages of 5 per lot

$$\text{Job mix tolerance}=\left(\frac{2.33}{3.00}\right)\times(A)\times(2.5)$$

$$=(0.78)\times(1.34)\times(2.5)=2.6$$

$\therefore$

UCL = 38.0 + 2.6 or 40.6 percent

LCL = 38.0 − 2.6 or 35.4 percent

Control limits for ranges were not established on this particular project. However, the central value and upper limit for 5 per lot would be computed as follows for the 99 percent level:

Range of 5 per lot

$$\text{Central value}=(d_2)\times(\text{standard deviation})\times(\text{reduction factor})$$

$$=(2.326)\times(2.5)\times\left(\frac{2.33}{3.00}\right)$$

$$=4.5\text{ percent}$$

$$\text{Upper control limit (UCL)}=(D_2)\times(\text{standard deviation})\times(\text{reduction factor})$$

$$=(4.918)\times(2.5)\frac{2.33}{3.00}$$

$$=9.5\text{ percent}$$

Statistically, the control chart for averages (fig. 3) was out of control as seven consecutive points were above the job mix target. However, on this job nothing was done; the situation probably was one of engineering judgment.

Several other States have or are proposing to use control charts. The Vermont Department of Highways, for example, plots all field test data on charts. The California Division of Highways is proposing to use a moving average of five as the basis for corrective action. Trends are more apparent with the moving average chart than with the average chart. Decisions on conformance of the sublot are based on individual-tolerance requirements for the test and on reduced-tolerance requirements for averages of the preceding four sublots plus the one under consideration. On the other hand, the Virginia Department of Highways proposes to plot only individual results and rely on individual probability trends for judgment of conformance. Initially, this approach is being applied experimentally to asphalt mixes. Kansas also plots individual bituminous extraction data and is starting to plot averages of the same data.

For the past 5 years, the Mississippi State Highway Department, the forerunner in the use of control charts, has been using a progressive-step chart to accept and control density of stabilized bases. The 1967 specification extends this use to other bid items. This seemingly complicated plan allows acceptance to be based on as few as two tests per lot, but a minimum of six tests are usually required for rejection. The plan has a built-in safeguard for excluding wild test results. In addition, the test results of the lot are averaged with all test results from the previous five lots, and this average must not deviate by more than a specified amount from the target value.

Summary

The use of control charts to regulate and accept highway construction is still in a fledgling stage. Up to now the industry has existed and expanded with the single test, retest, and engineering-judgment concept. The use of control charts will not alter the fact that the engineer has the final decision in accepting construction. The charts, however, are effective tools to visually forewarn that undesirable trends may be developing and to help both the engineer and the contractor decide *when to take action* and *when not to take action.*

The adoption of control-chart use by the highway industry would provide a significant technological advance in ascertaining adequacy of construction. Almost anyone can spot large changes, but successive small changes that develop as trends are not easily detected, even by experienced people. The picture portrayed by the control charts would highlight the trends and motivate the use of such charts as a decision tool.

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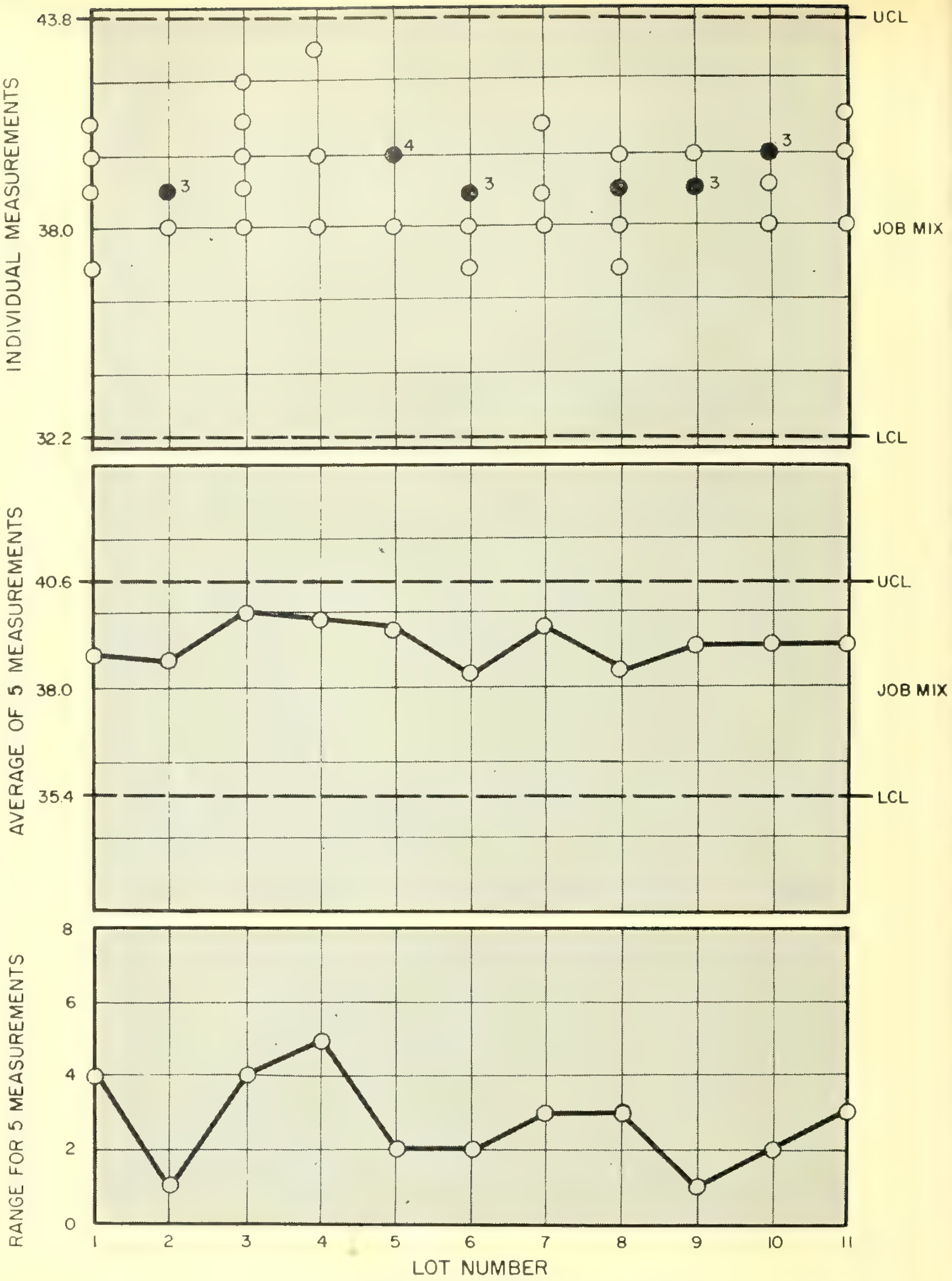

Figure 3.—Control chart used by South Carolina.

Fatal Accidents Involving Tractor-Trailer Combinations in Rear-End Collisions on Completed Sections of the Interstate System, 1968

Harold R. Hosea, Office of Traffic Operations, Bureau of Public Roads

The article *Fatal Accidents on Completed Sections of the Interstate System, 1968*, in the October 1969 issue of PUBLIC ROADS presented a general summary of the characteristics and circumstances of fatal Interstate-Highway crashes. Differences in the types of accidents and in the vehicles involved produce rather widely divergent patterns, and in this supplementary report one accident type, rear-end collisions involving tractor-trailer combinations, is examined in more detail.

In the general summary it was noted that tractor-trailer combinations were involved in rear-end collisions relatively more frequently than other types of vehicles, particularly passenger cars. Almost a third (31.6 percent) of the tractor-trailer combinations primarily responsible for fatal accidents were involved in rear-end collisions, compared with only 12.6 percent of the passenger vehicles. This comparison is based on the assignment of primary responsibility for the collision to one of the two or more vehicles involved, as determined by analysis of police investigators' reports. It does not necessarily imply any legal responsibility for the accidents. Vehicles considered to be primarily responsible for rear-end collisions are usually, but not always, the striking vehicles.

In the general summary, vehicles involved in the rear-end collisions, but not primarily responsible for them, were not discussed. A supplementary analysis of the detailed accident reports provides two additional types of information on these collisions: (1) the characteristics of the collisions for which drivers of tractor-trailers were primarily responsible, and (2) data on the rear-end collisions involving tractor-trailers for which the drivers of other types of vehicles were primarily responsible.

As indicated in the accompanying table, tractor-trailer drivers were primarily responsible for 80 or approximately one-fifth of the total rear-end collisions. Fewer than half of these involved passenger vehicles, and almost a third involved two or more tractor-trailers. This seems somewhat unexpected as tractor-trailers constitute only about 10 percent of the total traffic volume on the Interstate System. However, tractor-trailers tend to travel more frequently at night, and to the extent that this occurs, the proportions of these vehicles in the total traffic stream at night would be increased above the general average. Of all the fatal rear-end collisions on the Interstate System in 1968, 61 percent occurred in hours of darkness as compared with 53 percent for all types of fatal accidents on this system. Also, indications are that tractor-trailer drivers sometimes tend to travel in queues.

Of the 88 fatalities, 31 were occupants of the tractor-trailers responsible for the accidents, and another four were occupants of the combination vehicles struck. Property damage to vehicles, cargo, and roadside appurtenances averaged $10,916 per accident. This figure is affected somewhat by the fact that, in a few collisions, more than two vehicles were involved.

Investigating officers reported that seven of the 80 tractor-trailer drivers were fatigued or dozing and that two had been drinking—one of whom was obviously intoxicated. In seven of the 26 accidents involving two or more tractor-trailers, tailgating, or following the vehicle ahead too closely, was reported as a significant contributing factor.

Accidents in which another type of vehicle was responsible for rear-end collisions involving tractor-trailer combinations outnumbered by a substantial margin—113 as compared with 80—those for which tractor-trailer drivers were responsible. In four-fifths of the accidents in the former category, the drivers of passenger vehicles were primarily responsible for the collisions. Only three of the 139 fatalities were occupants of the tractor-trailers that were struck. Property damage averaged $4,106 per accident.

More than a fourth of the 113 drivers of vehicles that struck tractor-trailers were reported to be asleep or fatigued. Another 31 were described as having been drinking—eight of them obviously intoxicated. Speeding was reported in 52 of these accidents; half of them involved speeds in excess of legal limits—a few as much as 100 m.p.h.

Of course the investigating officers' reports of driver conditions and violations differ widely with respect to quality of reporting and completeness. Nevertheless, it seems significant that driver condition was reported to be a contributing factor in half the accidents in which drivers of passenger vehicles collided with tractor-trailers, compared with one-fifth of the collisions for which tractor-trailer drivers were primarily responsible.

Fatal accidents involving tractor-trailer combinations in rear-end collisions on completed sections of the Interstate System, 1968

Type of collision [1]	Total accidents	Fatalities			Property damage	
		Vehicle 1 [2]	Vehicle 2	Total	Total	Per accident
	Number	*Number*	*Number*	*Number*		
Tractor-trailer—passenger vehicle	35	4	48	52	$226, 600	$6, 474
Tractor-trailer—tractor-trailer	26	22	4	26	522, 200	20, 077
Tractor-trailer—single-unit truck	14	5	....	5	116, 000	8, 286
Tractor-trailer—pickup or panel	5	....	7	7	9, 300	1, 860
Total collisions, tractor-trailer responsible	80	31	59	88	873. 300	10, 916
Passenger vehicle—tractor-trailer	93	111	3	114	337, 500	3, 629
Single-unit truck—tractor-trailer	5	5	....	5	11, 600	2, 320
Pickup or panel—tractor-trailer	15	20	....	20	73, 800	4, 920
Total collisions, other vehicle responsible	113	136	3	139	422, 900	4, 106

[1] The vehicle listed first is the one primarily responsible for the accidents.
[2] Vehicle primarily responsible for the accidents—that is, the striking vehicle with a very few exceptions.

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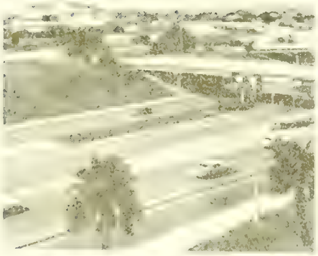

COVER

A major trunk route from the Gulf Coast to the Texas interior, Interstate Highway 37 also carries local traffic in Corpus Christi. The Corpus Christi Harbor Bridge can be seen in the background. (Photo courtesy Texas Highway Department)

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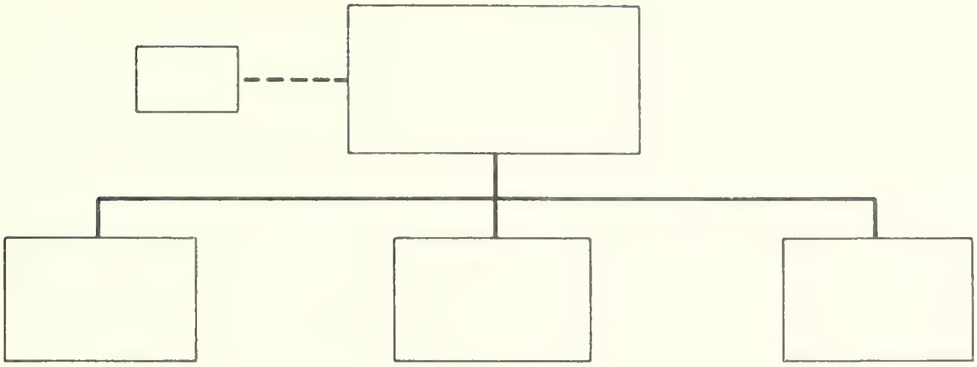

State Highway Department Management

Part 1.—Organization

Reported by [1] **PRISCILLA FAMOUS,**
Management Analyst
Economics and Requirements Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Beginning with this issue, the Public Roads research staff will report on management studies that were conducted in several States to upgrade the quality of highway administration. The findings that related to the organization, or structure, of the highway departments surveyed in the studies are reported in this part. Findings on other aspects of highway-department management will be presented in future issues.

It is hoped that all State highway departments will be motivated to modify and adapt those findings that may be suitable for specific application.

Introduction

A PUBLIC Roads program adopted early in 1963 enabled State highway departments to use a part of their Federally allocated highway planning and research (HP&R) funds to investigate the administration and management practices of their highway organizations. Research investigations were to be formulated by the States to yield results that had general, rather than specific, application. The findings and recommendations for improvement were to be documented for each project in written reports, from which the Bureau of Public Roads planned to prepare a final generalized report. In response to the program, research projects were approved for a number of States.

Because the final reports from these projects originated from 11 different sources, each with its peculiarities and specific problems, preparation of generalized summaries for all areas of the research was impractical, and it was concluded that instead of a single document, the Public Roads report would consist of several parts, each dealing with a particular segment, or area, of highway-department management. The first part, which deals only with those findings and recommendations that concerned organization, or structure, of the highway departments surveyed, is presented here. In addition to the information from the State reports, (*1-11*) [2] information on *types of organization* was obtained, from the Highway Research Board (*12*) and from other sources. When later information was available, the original information obtained from the State reports was updated prior to publication, and when administrative or legal actions were taken at the State level to carry out the recommendations of the State reports, they were noted if the author was aware of them.

The principal purpose of any formalized organization, or structure, is to provide a framework to make decisions and to execute them. The authority to make decisions and execute them must be delegated, and in the laws of each State, the authority to administer roads and highways is delegated to a single executive, to a highway board or commission, or to a combination of the two. The State law also may provide for the appointment of the chief engineer, treasurer, and other staff personnel, and may designate the functional divisions through which highway operations are carried out, but the internal organization of the department, generally, is left to the discretion of the single executive, board, commission, or director.

In most States, it is a legal requirement that the State's attorney general be the highway department's legal representative. Thus the highway department's legal division is an extension of the attorney general's office, and the department's legal officer, if not appointed by the attorney general, must receive his approval. Other legal restrictions on many of the highway department's responsibilities for the highway system are in the areas of personnel, purchasing, and administration of budgeted funds. Negotiations with other State agencies often involve unraveling complexities that result from out-moded procedures, which cause delays in placing needed personnel in operations and in programing. It was the opinion of one consultant that, to counteract such impedances, the other State agencies should establish policies that would allow the highway department, as well as other State departments, to act independently in its execution.

With respect to highway department organization, the Federal law states that before Federal aid for highway projects will be approved, the department shall have adequate powers, and be suitably equipped and organized to discharge to the satisfaction of the Secretary the duties" that the law requires, and specifically provides that the organization include a secondary road unit. (USC Title 23, sec. 302.)

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[1] Prepared for publication by David Summerville following the death of Miss Famous.

[2] Italic numbers in parentheses identify the references listed on page 287.

Types of Organization

Included here is a review of the generally recognized types of highway organizations that are used in many of the States.

Single executive

Under the single executive type of organization, highway administration is directed by one person; no boards or commissions of any kind are included in the organization. In some States the statutes contain a general grant of authority that permits the executive to exercise all the powers and perform all the duties of the department; those powers and duties that are specifically enumerated are then vested in the department. In other States specific powers are granted directly to the executive, as in Alabama where the State law reads, "The chief executive officer of the highway department shall be known as the highway director . . . all the powers, authority, and duties in the highway department shall be exercised by the highway director." (Code of Ala., Title 23, sec. 1.) Almost all the specific powers granted in subsequent sections are vested in the department, which, of course, is an indirect way of vesting them in the director.

Single executive with advisory board or commission

In several States, direction is provided by a single executive and a multimember advisory board or commission whose purpose is to advise the executive in carrying out his responsibilities as the directing head of the highway organization. Sometimes, the advisory bodies are assigned specific duties. They may make surveys to determine the conditions and needs of highways in their respective areas of the State, may give advice on the establishment of broad policies, or may perform many other necessary functions, as directed. In effect, the board or commission has no administrative authority, as may be stated by the statutes. One State in this category is West Virginia where, under the laws of this State, the State road commissioner is the chief executive officer of the commission and has direct and complete control of the entire State road program and system. The State road commission is designated an advisory body to the commissioner. (W. Va. Code, Ch. 17, secs. 16–2A–1, 17–2–1.)

Single executive and commission in coordinate capacities

In other States, the law concerning highway administrative direction divides the authority between a single executive and a commission, each acting in a coordinate capacity to the other. The commission is a part-time body, and the single executive is a full-time official who is designated commission chairman and chief administrative officer of the highway department and who is charged with the duty of executing all orders and decisions of the commission. The powers and duties vested in the commission are fixed by statute. The laws of Maryland can be interpreted as having this form of organization. They provide for a seven-member State roads commission consisting of six part-time members, appointed from specified areas of the State, and one full-time member, appointed from the State at large. The at-large member serves as chairman of the commission and is also designated director of highways.

Generally, the director is responsible for Interstate and primary highways, and the part-time members for secondary highways. Specifically, the director has "exclusive direction and control over the location, construction, geometrics, design, and maintenance" of Interstate and primary highways and "complete jurisdiction and authority over the engineering and right-of-way departments of the commission and the entire administrative organization of the commission itself." (Annotated Code of Md., Article 89B, sec. 1–7B.) The part-time members of the commission conduct a continuing survey of the State secondary highways in their respective areas, and the full commission formulates policies relating to location, type, design, construction, and maintenance of the secondary system. The commission also determines the extent that consulting engineers will be employed on the entire State highway system, but the director has the sole authority to select all consulting engineers employed.

Limited-control commission

Under the limited-control commission form of highway administration, the statutes provide for a part-time commission and for the appointment, usually by the commission, of an executive administrative officer. The law may require that the commissions limit their duties to the determination of policy and policy-related activities. Thus the executive officers, although subordinate to the commissions, are left free to administer the departments.

Arizona's law provides that control of the highway department be vested in a highway commission, but it further states that the commission shall "in general exercise powers and duties which relate to adopting and carrying out policies of the department and control of its financial affairs." Another section of the Arizona law reads as follows: "The director shall be the chief executive and administrative officer of the department and shall have charge of the administration of all highway affairs authorized by the commission." (Ariz. Rev. Stat. Annotated secs. 18–106, 18–109.) In general, then, the Arizona law limits the commission's functions to policy determination and related duties and gives the director authority to perform administrative functions.

Administrative commission

Commissions that exercise considerable administrative control, as distinguished from those that are restricted to policy determination or perform only limited administrative duties, are administrative commissions. The members of these commissions usually serve full time. Highway operations in Maine are organized under an administrative commission, a three-member State highway commission, in which only the chairman devotes full time to commission duties. According to the law, "The chairman shall be the chief administrative officer, having general charge of the office and records, but all policy decisions of the commission must be by a majority of its total membership." (Rev. Stat. of Maine, Ch. 23, sec. 3.) On the basis of the salaries paid, it is assumed that the other members of the commission are expected to devote considerable time to their duties. On the other hand, the three administrative commissioners in Wisconsin each devote full time to their duties, and the administrative and executive authority is vested in the chairman. (Wis. Stat. Annotated sec. 84.01.)

In all the States, except Georgia, Mississippi, Nevada, and South Carolina, statutes pertaining to highway department organization provide for the governor to appoint commission members and single executives and, generally, for one or both bodies of the States' general assemblies to confirm the appointments. In Georgia and South Carolina, highway commission members are elected by the respective general assemblies. In Mississippi, the citizens of the State's three supreme court districts elect, by popular vote, a commissioner for their district. In Nevada the board of highway directors consists of the governor, attorney general, and State controller, all elected officials.

Departments of Transportation

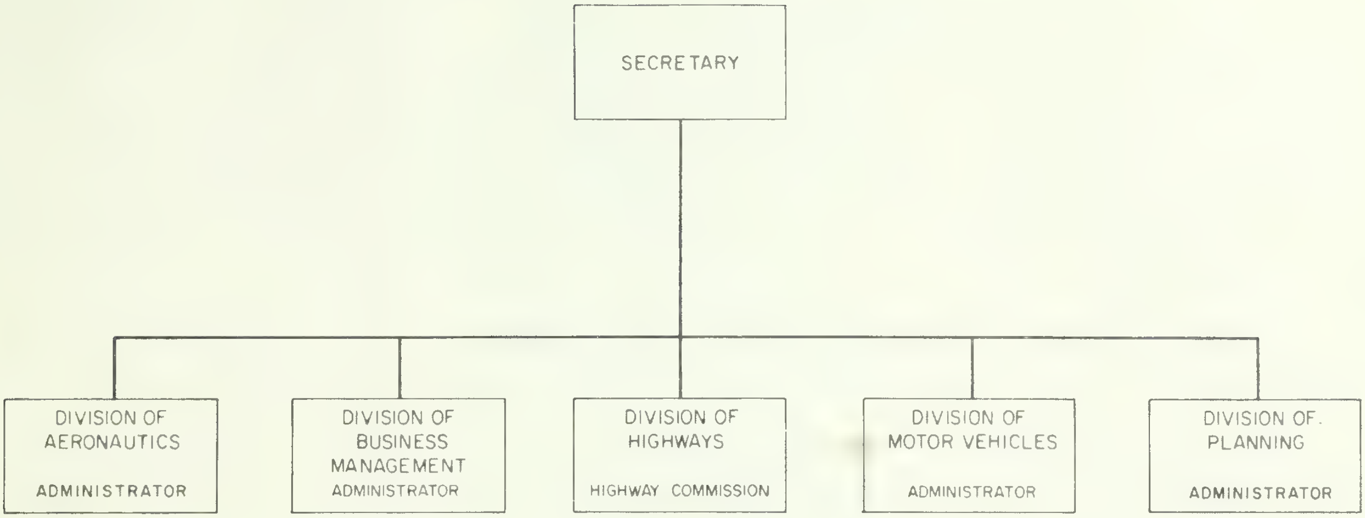
One of the newest forms of organization for highways is to include the highway department in a State department of transportation. Hawaii, the pioneer of this type of organization, introduced a department of transportation as a State agency when Hawaii was admitted to statehood and reorganized its government. Following is a brief description of the department of transportation in Hawaii and of those in the other States that have since established them:

Hawaii

The first legislative assembly in 1959, by enactment, provided for a department of transportation to be headed by a single executive titled director. The new agency incorporated all the functions, duties, and powers, of the former aeronautics, harbor, highway, and public-works commissions. The director of transportation is appointed by the governor. The law provides for the establishment, within the department, of a commission on transportation "which shall sit in an advisory capacity to the director of transportation on matters within the jurisdiction of the department of transportation." (Session laws of Hawaii, First State Legislature, Second Special Session, Act 1, 1959.)

California

In 1961, California established what is now titled the Business and Transportation Agency; the original title was Highway Transportation Agency, but the statute was amended in 1965 to remove the word "highway" from the title. The Agency includes the former departments of motor vehicles; highway patrol and public works, which includes the division of highways; aeronautics, contracts, and rights-of-



Organization of the Wisconsin Department of Transportation.

way; and bay toll crossings. The law provides that the transportation agency be headed by an administrator who is appointed by the governor and is designated chairman of the State highway commission; however, legislation approved in 1967 removes the administrator from the highway commission and provides for selection of a chairman by majority vote of the commission. (West's Annotated Calif. Code, Government Code, sec. 13975; and session laws 1967, S46.)

New Jersey

The department of transportation in New Jersey was established in 1966 as a principal department in the executive branch of the State government. It provides for the appointment by the governor of an administrator titled commissioner of transportation. All the functions, powers, and duties of the State highway department and State highway commissioner ,of the division of railroad transportation, and of the department of conservation and economic development and its commissioner were transferred and vested in the department of transportation. (N.J. Laws of 1966, sec. 413.)

Another unit of the new department is the commuter operating agency, which is charged with unraveling problems of dwindling commuter rail passenger service and with studying the possibility of changing the routes, services, and fares of the commuter lines that have been limiting their services for some time.

New York

The legislative enactment establishing a department of transportation for New York became effective September 1, 1967. The head of the new department, designated commissioner of transportation, is appointed by the governor for a term that is concurrent with the governor's term.

The department is charged with the formulation of a long-range, comprehensive, statewide transportation plan for the balanced development and coordination of adequate, safe, and efficient commuter and general-transportation facilities and services in the State at reasonable costs to the people. These transportation facilities include highways, rapid transit, railroad, omnibus, marine (inland waterways), and other transportation facilities and services, as well as aviation and airport facilities and services, whether publicly or privately owned, developed, operated, or maintained. The new law abolished the former department of public works. Except for the functions of flood control, shoreline and beach erosion control, and the construction of public buildings, which were transferred to the office of general services, and to the conservation department, all the functions powers, and obligations of the public works department were transferred to the department of transportation. (McKinney's Session Law News, No. 5, May 25, 1967, and Regular Session of 1967, Ch. 717, titles 5–8).

Wisconsin

Statutes enacted by the 1967 Wisconsin legislature provide for reorganization of the executive branch of the State government. As a part of this reorganization, a department of transportation was created. The agency known as the State highway commission was renamed the division of highways and transferred intact and attached to the department of transportation. The head of the new department is a secretary of transportation who is appointed by the governor. The law provides that any agency attached to a department shall exercise its powers, duties, and functions prescribed by law, but that budgeting, program coordination, and related management functions shall be performed under the direction and supervision of the head of the department. (Laws of Wis. 1967, Ch. 75, effective August 1, 1967).

Florida

The Florida law, or House Bill No. 685, passed by the 1967 legislature, established a transportation commission, a department of transportation, and a transportation authority. The chairman of the State road board is an ex officio member of the three-member transportation authority. The authority is an advisory board to the transportation commission whose chairman is the governor. The responsibility of the authority, other than being an advisory unit, is to coordinate the functions of the public service commission, State road department, and the department of transportation. No change was made in the authority of the State road board, or in its organizational status in the State government.

Delaware

Although the organization in Delaware is listed frequently in publications as a department of transportation, the term has been applied to what is generally considered mass transit and the State highway department is not involved.

Other States

During the 1967 sessions of the general assemblies in several States, bills to create transportation departments were introduced. The West Virginia bill came to vote but failed passage. Six States passed bills that authorized studies prior to establishing departments. During the 1969 sessions, Colorado, Connecticut, Maine, New Hampshire, Oregon, and Washington introduced bills to establish transportation departments, and other States gave consideration to such bills.

Six States have highway departments that were established by constitutional enactment rather than by legislative enactment. These States are Arkansas, Georgia, Louisiana, Michigan, Missouri, and New Mexico. Usually, highway department organizational changes are more difficult to make under constitutional establishment than under legislative establishment.

Organization and Management of State Highway Departments

The findings from the organization and management surveys of State highway departments conducted under the HP&R projects by consultants in 11 States, are discussed in the following paragraphs. In presenting the information, each State is treated individually, as no uniform type of highway organization exists. Although alike in many details, highway organizations differ among States largely because size, population, political development and other factors have a bearing on the type of organizations used. The purpose of a department organization is to provide a form, or order, through which those concerned with management can operate. In general terms, highway management operations can be classified as top, divisional or staff, and field, and each is discussed as reported in the individual project reports.

Top Administrative Organization

Commissions and directors as top administrators and the legal bases for organization were discussed under the heading *Types of Organization*. In addition to commissioners and directors, chief engineers also may be considered top administrators. The recommendations concerning top administrative organization, as they apply to specific States follow:

Indiana

In Indiana, it was recommended that both the role of the highway commission, as the policymaking body, and that of the highway director, as the chief executive officer of policy with authority and responsibility for the internal affairs of the department and highway program, be clarified and established in the law to prevent members of the commission, either individually or as a group, from becoming involved in operational matters. It also was recommended that as the commission by law is responsible for all highway matters, it should be empowered, rather than the governor, to appoint the executive director to serve at its pleasure. It was futher recommended that written guides, describing the functions, responsibilities, authority, and relationships with other organization units, be prepared for all top management positions so that personnel filling the positions will know the duties and activities for which they are responsible and accountable.

Washington

As for Indiana, it was recommended that the roles of the highway commission and the highway director in the State of Washington be clarified in the law to restrict operational matters solely to the director. Other suggestions and recommendations for the top administrative level in the State of Washington were:

- That the highway commission give its time and attention to what was termed "basic concepts of boardmanship," as follows:
  (1) the commission should require that detailed objectives be developed for each phase of the department's operations. Objectives should be both short- and long-term, be clearly stated in writing, and be periodically reviewed;
  (2) the commission should stimulate and oversee the development of short- and long-term plans designed to achieve objectives;
  (3) the commission should establish basic departmental policies that are broad and flexible enough to give clear guidance to departmental staff and eliminate rigidity that restrains imaginative administration;
  (4) the commission should develop criteria against which the performances of the department and highway director can be evaluated; and
  (5) the commission, together with the director, should be responsible for developing a program of outside relations with the legislature, with other State and Federal agencies, and with the public.
- That the executive officer be formally authorized, either by law or policy, to delegate to subordinates, as he considers necessary, his duties, responsibilities, and authority.
- That a monthly meeting of the commission, governor, and director be scheduled to improve relations between the highway department and the governor, who it was observed was not kept informed of highway operations, and whose views on matters of policy were not always considered. At this meeting, the activities of the department could be discussed and the views of the governor on policy matters ascertained.
- That the director, as chief executive officer and ex officio member of the commission, develop and keep current a comprehensive policy manual; that he develop a new agenda format to conduct commission meetings more effectively; and to relieve him of administrative detail and allow him more time for highway matters, that he eliminate interdepartmental correspondence, document signing, etc., being routed through his office.
- That a management staff, consisting of the assistant director and the heads of the major operational functions, be formed to meet regularly with the director to discuss problems and report on program and project status.

Nebraska

Reportedly, one purpose of the survey in Nebraska was the development of a program "to effectively plan and manage the State's overall highway program on a coordinated statewide basis." Accordingly, both State and local highway legislation, organization, staffing, and operations were analyzed. Recommendations were made to correct weaknesses or problems found in these areas, many of which were believed caused by retention of an organizational concept that was inadequate and incompatible with today's advance in cultural, technological, and economic facets of society. It was concluded that the intent of the legislature to vest in the governor and highway department the responsibility for planning, constructing, and maintaining the State highway system is adequate and provides on a statewide basis the necessary framework for effective highway planning and management. Safe and efficient highway transportation is of interest to all the people of the State, and an integrated system of highways is essential to the general welfare. For the purposes of the management study, this statement was interpreted as authorizing the formation of a State-level organization the primary objective of which is to provide an integrated system of highways, roads, and streets, and as defining the responsibilities of the highway organization to include administration, planning, design, construction, and maintenance of such a system. It was the opinion of the consultant, that "implicit in the objectives," as contained in the legislative intent, "is the responsibility for the Department of Roads to provide leadership, guidance, and control recommendations to all levels including county, city, and local governmental units," concerned with highway operations, but that the "attitude of the Department of Roads appears to be that it does and should confine its concern to matters affecting the specific system defined as the State highway system."

To effect the development of an integrated system of highways, and also to determine what functions should be performed at the several levels of government, the consultants believed that all laws, policies, and procedures should be designed to force authority and responsibility to the lowest level possible, consistent with good management and efficient operation. Thus, the local political units should assume responsibility for performing, with a minimum of control, the day-to-day activities associated with highway planning, design, construction, maintenance, and financing of their highways, roads, and streets; if these local units will not accept it, the responsibility should be transferred to the next higher level. Final or central authority must be vested in the State highway department and include the power to prevent or discourage local units from using funds and pursuing policies that are hostile to overall development of the integrated highway system. Such authority, it was pointed out, can be exercised by withholding funds, by selecting and dismissing personnel, and by granting incentives for the attainment of objectives. Highway-user allocations and local tax levies for the road system should be considered State funds rather than local funds, and, as such, should be used to develop a system of highways, roads, and streets to benefit all citizens of the State.

For the highway department to exercise its authority and assume responsibility for an integrated highway system, it would be necessary, particularly where local governments are conerned, to enact additional legislation or modify existing legislation as follows:

- Revise the section of the law termed *legislative intent* to mean that the department through its director is to formulate short- and long-range needs for highways, roads, and streets to provide for an integrated system of roadways and bridges, to provide administrative and technical assistance on highway matters to counties and cities, to promote and coordinate cooperative planning ande ffort between governmental units, and to organize and staff a department at State level to provide for an integrated highway system.

- Define the statewide road system, incorporating in the definition the intent of the legislature to provide an integrated system, and assigning to the department administration of the system and responsibility for it.

- Require the department to develop a functional classification system, and assign all highways, roads, and streets to one of the classifications.

- Assign to counties and municipalities the administrative responsibility for highways that are classified *local*. No responsibility should be assigned to a unit below the county level. Encourage municipalities to contract with counties or vice versa to provide highway services when the good of residents can be served through integrated, economic operations.

- Assign specific planning, design, construction, maintenance, and financing responsibilities to each highway classification. Assignment should specify the services, such as maintenance, that the local government is to finance through local tax revenue only.

- Assign to the department responsibility to develop, document, and provide means for implementation, standards of design, construction, and maintenance, to guide local units in the discharge of duties assigned. Provision for monitoring use of standards by the department should be included.

- Require the department to develop and implement a plan to identify and update biennially long-range needs of all highways, roads, and streets, and to use the resultant value of needs as a guide for distribution of State-aid to local government units.

- Require the appointment in each county and municipality of an individual with qualifications approved by the department, and assign to him the following duties:
  (1) Develop and submit to local units long-range plans, updated annually, based on needs; annual programs for design, construction, and maintenance; and annual budgets based on programed projects and activities. Submit, after local approval, plans, programs, and budgets to the department.
  (2) Direct operations of State-approved activities; certify upon completion of projects that State-established standards have been complied with, and recommend that work done by contract be accepted and the contractor paid.
  (3) According to the system and schedules developed for the purpose, prepare and submit annually to the State auditor and to the department a report showing actual receipts, disbursements and accomplishments compared with those budgeted and programed. Include provisions that the State auditor and department have authority and responsibility to withhold State aid when standards established for such report are not complied with.

- Incorporate provisions to encourage two or more local units to appoint the same individual when operating economy can be assured. The term individual should include consulting engineers and firms on less than full-time basis. Require that registered engineers certify and approve all construction projects for counties and municipalities.

- Assign to the department authority and responsibility to work with counties, municipalities, and other subdivisions to develop an integrated road system; the department must have final authority should differences of opinion occur. Also, assign authority and responsibility to establish safety codes, sign structure, driving rules, etc., on the road system.

- Develop incentives for counties and municipalities to adopt centralized organizations and to consolidate them when economically feasible. Paying part of the salary of an engineer to supervise road and street operations was one incentive suggested.

- Establish a permanent transportation committee consisting of legislative members to evaluate highway needs, to determine effectiveness of highway programs, and to consider future legislation.

- Eliminate all duties, now assigned to the head of the department, that are not directly related to the activities of the department.

- Abolish the State highway commission and establish, as the intent of the legislature, that the governor and department head shall have full authority and responsibility for the conduct of the department. The consultants concluded that the commission lacked authority to do anything but give advice to the head of the department and to the governor—and only at their request—and be listener to complaints against the road system and department. To meet the need for group thinking at the policy-forming level, the consultants recommended the formation—as an integral part of the department—of a management committee made up of six department members and three public advisors. Members would be the department head; the directors of administration, planning, design, and field operations; and the public relations manager. The department head would be the chairman, and the public relations manager the secretary. Selection of the public advisors would be the responsibility of the department head; the advisors would serve at the department head's discretion, and be outstanding, knowledgeable individuals—one each from the financial, managerial, and promotional areas of commerce and industry. Suitable fees and expense allowance should be provided the public advisors, and monthly meetings of the management committee should be considered.

Existing law provides for a department head, but does not give him a title although giving him "full control, management, supervision, administration, and direction of the department, and all powers and duties conferred on the department are exercised under his direction."[3] There is no statute authorizing any one individual or group as the policymaking authority. It is implied that the head should set policy as well as administer and that final authority is with the governor. The legislature, it was concluded, lacked the necessary technical skills to make highway policy and, furthermore, did not convene often enough to be responsive to the department's needs.

The consultant's analysis of the highway department's organizational structure establishes the head of the department as the administrator. Staff functions reporting directly to the administrator are audits, personnel, program and planning, information, and, to some extent, legal. The administrator is also responsible for adopting and enforcing the rules and regulations pertaining to the duties of the State's safety patrol and for maintaining an office for law enforcement and public safety. The technical and engineering services and the field divisions report to the administrator through a deputy administrator. All of the staff groups, whether reporting to the administrator or to his deputy, have been delegated authority and responsibility in their respective areas for direct supervision of field offices on matters concerning their staff functions. The administrator's or deputy's contacts with the district offices are usually to resolve disagreements between staff heads and district engineers rather than to exercise management leadership. This supervisory arrangement organizationally isolates the district from the department.

According to the analysis other obstacles to effective organization and management are as follows:

- Either there is an apparent lack of well-defined policies, which restricts decisionmaking and leads to inconsistencies, or decisionmaking was so consistent that initiative and new ideas were being destroyed.

- Management is involved in details that are more appropriately delegated to subordinates. Long accepted practices are difficult to change or modify; no one wants to risk changing what is accepted.

Alabama

The general criticism of the Alabama highway organization, as reported by the consultants, was that functional sections were expanded in the organization without giving sufficient care to their placement, and also that legal and political considerations left management little discretion for placing functions and positions.

Organized under the single executive type of management structure, the director, as well as being the policy maker and designated chief executive officer, had four functional units reporting to him in addition to the chief engineer and the executive assistant. In a reorganization plan that was developed, only the director's staff—executive assistant, legal services, and chief engineer—were placed under his direct supervision.

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[3] Following the recommendations the department head was given the title Director, State engineer, to indicate his responsibility.

In the opinion of the consultants, the position and responsibilities of the executive assistant needed clarification. This official, it appeared, had direct lines of contact to all personnel in the department, shunned established lines of responsibility and communication, thus staff levels of management were not always fully aware of the activities in their subordinate organization.

For the more effective handling of its legal matters, it was suggested that the department have its own legal staff. Although the legal staff would be legally responsible to the attorney general, it would report to him through the highway director. In addition to providing better legal facilities, this arrangement would reduce legal expenses through Federal participation in many legal costs.

It is apropos to note here that the plan proposed for reorganization was used as a basis for a management structure adopted in November 1963, and recommendations for the improvement of legal activities later were enacted into law. The law creates a legal division for the department, to be headed by a chief counsel appointed by the highway director with approval of the attorney general. The director was also authorized to appoint, with the attorney general's approval, and subject to the State's merit system, assistant counsel as necessary to transact the department's legal business; appointees were to be full time and paid from highway revenue.

Minnesota

The survey of the Minnesota State Highway Department, outlined these criticisms of the organizational plan: Lack of flexibility in meeting changing work requirements; inappropriate balance between engineering and management skills; some programs overemphasized at the expense of others of equal importance; related activities dispersed; and an inappropriate division of functions and responsibilities between the districts and the maintenance areas. It was recommended that a plan for organization be structured around the department's objectives and the nature and character of work required to achieve them.

In this State, the overall leadership is vested in a State highway commissioner who is authorized, by law, to organize the department, employ personnel, and delegate to subordinates the responsibilities that he considers, in each instance, necessary to carry out the department's duties and functions. In addition to the commissioner's staff, as provided by the heads of the several functional divisions, it was proposed that the commissioner appoint a public advisory council of 12 to 15 members to keep him informed as to whether his policies and programs are meeting the expectations of the people in different sections of the State.

The State has implemented most of the consultant's recommendations, but benefits are difficult to measure. The department has been restructured under six major divisions, and as a result, the organization is flexible enough to meet future needs, has a more manageable span of control, has clear cut lines of authority and responsibility, and makes provisions for increased delegation of authority to field districts.

The top management group has been redefined to include the commissioner, deputy commissioner, one deputy commissioner for operations, and five assistant commissioners, and their roles in program formulation and execution have been specified.

Benefits from the recommended reorganization of external relations has been marked. Progress has been made in the areas of data processing, field maintenance, drivers license, and equipment maintenance, with resulting cost savings.

More effective approaches to government and community relations have been adopted to provide improved relations with the legislature and help to untangle red tape with the State department of administration.

Missouri

In Missouri, the authority for highways is placed in a part-time policymaking commission. The chief administrative officer is the chief engineer whose time must be divided between meetings with the public and highway department activities. Often, he is not present when decisions that cannot be made at a lower level of management are required. The assistant chief engineer is responsible for certain divisions and can only investigate and pass on to the chief engineer the problems of other divisions. At the next lower level are two assistants to the chief engineer who have not been delegated adequate authority to correspond with their areas of responsibility. Each must await the return of the chief engineer and vie for his time when a decision is needed or a controversial problem is encountered. It was pointed out in the study that some realinement of the highway department's top management organization or structure appeared necessary to segregate and place responsibility for decisionmaking, planning, execution, followup, and control.

Since the recommendation was made, the commission announced that it will appoint an administrator to serve under the direction of the commission. The administrator will have general charge of the department and will perform such duties and have such authority as the commission may designate.

Virginia

In Virginia, the law provides that the highway department be administered under a part-time commission with a full-time chairman designated highway commissioner. He is charged with the execution of all orders and decisions of the commission and is vested with plenary powers for constructing and maintaining the State highway systems. The statutes define the powers and duties of the highway commission, but the authority and responsibility for setting policies and objectives of the department is not included among them. To overcome this deficiency and to induce commission members to take a broader-than-district view of objectives, it was recommended:

- That members continue to be appointed from the construction districts into which the State is divided, but that they be directed by statute to represent only the broad interest of the State as a whole and not be guided by the interest of their home districts. This recommendation was enacted into law and became effective June 1964.
- That the highway commission be given authority and responsibility to approve and censure the administrative management of the department. This recommendation also was enacted into law and became effective June 1964.
- That the term of office for highway commission members be increased from 4 to 6 years, and that the members organize themselves into several subcommittees. The proposed committees were administration and public relations—to consist of three members with the director of administration serving as secretary; planning and finance—to consist of three members with the director of programing and planning serving as secretary; and policy—to consist of two members with the director of operations serving as secretary.

A reorganization plan formulated to overcome other weaknesses observed in the management of the department recommended:

- That the State highway commissioner be designated chief executive officer for administration, and that the chief engineer be designated chief executive officer for operations, to relieve the commissioners of operations details.
- That a new echelon of management—four directors—be created to relieve the concentration of responsibility in the top managers. This would add two more top executives and enable the grouping of related divisions. The new officials and the divisions under their supervision are: Director of programing and planning—to include right-of-way, research, traffic and planning, programing, and scheduling; director of engineering—to include bridge, location and design, materials, secondary roads, and urban; director of operations—to include construction, equipment, maintenance, landscape, and field districts; and director of administration—to include fiscal and accounting, personnel and organization, purchases and stores, toll facilities, and public information. The director of administration is to report to the highway commissioner, and other directors to the chief engineer–deputy commissioner. This recommended management plan was used as the basis to reorganize the highway department in 1965. It reduces the number of divisions heads reporting directly to the highway commissioner, and gives him the opportunity to devote his time to the executive responsibilities of his position. It also establishes formal channels of communications between the several divisions and the field offices.

In discussing State highway organizations generally, it was stated that the legislatures in all States provided for the administration of highways and, in some States, set up in law a skeleton organization. The skeleton organization action is not always considered to be advantageous, as most law makers are not experienced or knowledgeable in administration and management. Under such conditions the administrator must organize the functions, employ the personnel, and issue the orders, always mindful of the powers and restraints imposed by legislative enactment and sometimes by the constitution.

Mississippi

The law in Mississippi provides for a popularly elected highway commission. The terms of the members are concurrent with that of the governor. The statutes also provide what may be termed organizational restrictions, as follows:

- The amount of salaries and maximum expenses allowed the commission are set.
- The commission must employ a director who is designated the chief administrative officer with the authority, under the commission and in conformity with its orders, to approve bids; sign all vouchers, requisitions, contracts and agreements; and order all supplies and materials. The director must employ, subject to the approval of the commission, a chief engineer, an engineer of construction, an engineer of maintenance, and a bridge engineer, and such other department heads as may be advisable and necessary.
- The chief engineer is legally responsible, subject to the approval of the director, to appoint and remove the district engineers and other assistants or employees of the department other than the engineers appointed by the director.
- The State-aid division, established "within the State highway department," is headed by an engineer appointed by the governor. He is authorized to employ and remove at any time assistant State-aid engineers and other engineers, assistants, and employees as necessary to carry out the purposes of the division.
- The heads of the several staff divisions and of the districts are required to be engineers.

Otherwise the commission and director may organize the department, establish pay schedules, employ personnel, and do whatever is necessary to construct and maintain the State highway system.

Another legal requirement that, in the opinion of the consultants, caused many of the highway department's management problems is the allocation of highway construction funds equally among the districts from which the highway commissioners are elected, which gives rise to a concept that there are as many State highway departments as there are commissioner districts. From the management point of view, this arrangement serves to force programing of construction activities within the districts, as opposed to programing of construction for the State as a whole, and tends to prevent management from developing effective and efficient programs for the entire State's highway needs.

Maintenance funds, also affected by this concept, are apportioned on the basis of mileage under maintenance, and the commissioner district that has the minimum construction cost per mile will increase its improved maintained mileage at the fastest rate, which results in increased maintenance fund apportionments for that district. Also, districts tend to treat machinery and equipment purchased with their maintenance apportionment as exclusively their own, rather than as assets of the highway department. Thus there are wide variances in standards of maintenance and in utilization of equipment and personnel, as well as imbalanced inventories and nonuniform reporting and control procedures.

The three-highway-department concept is also a problem in accounting for the construction funds. To comply with the statutes, the department must establish separate funds for each commissioner district, and, as an operating procedure, separate funds also are established for each construction district within the commissioner district. Such accounting procedures add to the costs of department administration. Furthermore, the economy of the whole State is affected, for it can best be served if highways are built where and when users demand them—not as construction fund apportionments restrict them. The principal organizational deficiencies found by the consultants are as follows:

- No provision was made for anyone to act in the director's absence, or for an interim administrative authority should prolonged illness or other incapacity remove the director from duty. Nor was there a provision for delegation of authority and responsibility to lower levels of employment. It was pointed out that the cause probably was the interpretation placed on the statutes, as nothing was found in the law to prevent the delegation of many of the director's duties to proper subordinate units.
- No provision was made for well-coordinated work programs between the several related staff functions. All the divisions and districts independently reported directly to the chief engineer and the director. There was no intermediate administrative-executive level to coordinate the work of related functions.
- Central office divisions and personnel were housed in quarters that were inadequate and at several different locations. Some functions were not assigned to the division that had primary operating responsibility, partly because of the lack of housing facilities, and partly because of personality and capability conflicts that could exist in any organization.

A pattern for reorganization of the highway department to eliminate these deficiencies was recommended. Although it was realized that the plan might never be fully achieved, it was believed that technical changes in construction and maintenance methods, improvement in administrative techniques, and the capabilities and personalities of employees will work against the highway organization's ever becoming static. The plan offers the following recommended goals:

- Establish the chief engineer as deputy highway director to act for the director during his absence, but only on the written authorization of the director and with the formal approval of the commission.
- Reassign functions under the most logical and most appropriate staff divisions. Establish separate divisions to administer personnel, procurement, office services, and equipment-management functions.
- Establish separate offices for systems and procedures and internal auditing, and make them staff functions of the director. These offices, working together, are to be responsible for designing and maintaining all policy and procedures manuals, designing and controlling all forms and reports, designing and installing all data collection and analysis systems, making all internal audits, and insuring compliance with department policies by the divisions and districts.

Two changes that would require legislation are: (1) Place the statewide division, which is now required by law and which is primarily concerned with the secondary road system, completely under the highway commission, and amend the law accordingly, and (2) amend the legislation requiring the chiefs of all functional divisions and the districts to be engineers. Where engineering knowledge is not required, the best interests of the department could be served if persons with other professional qualifications acted as division chiefs. However, it was pointed out, that there is no reason why an individual trained in the engineering sciences cannot, through experience, develop adequate capabilities in administration and management. Some functions or divisions that would not require administrators with engineering knowledge are personnel and merit system, financial control, right-of-way, office services, equipment, and procurement; appointments should be left to the discretion of the commission and director.

After examining alternative methods to establish proper administrative controls, the consultants included in a new State operating procedures manual those methods that appeared most practical for the department and submitted the manual to the highway commission for approval. This manual replaced the administrative memorandum series and the administrative procedures developed through interoffice letters, or independently, by each responsible division or district. The manual appeared to be written to correct misunderstandings rather than to prevent them.

The principal recommendations that did not require legislative action became effective in Mississippi as of July 1965. A manual containing policy statements, established areas of responsibility and authority, organization charts, flow charts, and exact copies of forms and instructions was compiled, and a systems and procedures section was established to keep the manual current. The chief engineer now acts during the director's absence. Internal auditing and public relations, formerly staff functions of the director, are now operating sections.

One recommendation, the construction of a highway department central office building, has not been implemented as the legislature has not authorized the expenditure. No changes in Mississippi law have been reported.

South Carolina

The laws of South Carolina provide the basic organizational structure for the highway department, and require the appointment of a State highway commission and the selection and appointment by the commission of three officers—a chief highway commissioner, as administrative and executive head of the department; a State highway engineer; and a secretary-treasurer. The law also provides that the department be divided into such divisions as the commission or the chief highway commissioner may prescribe, but states that it shall consist of at least these principal divisions: Engineering, motor vehicle, and law enforcement. The latter two may be combined under one director.

Legislative declaration or statement of policy is one feature of modern highway law, and such a statement should be given consideration for inclusion in South Carolina's existing statutes. The declaration should be clear, concise, and written to inspire the administrative body and to encourage maximum use of technical and management skills and resources to achieve the stated objectives and policies.

It was suggested that the head of each division be designated director, and that some standard designations be adopted for the subdivisions. Standardization of unit designations would aid organizational control and position classification, and prevent unofficial acquisition of status and title.

Indiana

The highway affairs of Indiana are vested in a State highway commission, which is a policymaking body having four members appointed by the governor for a term of 4 years. The law provides that the members serve at the pleasure of the governor who also is authorized by law to appoint an executive director of highways—not a member of the commission—who is designated the chief executive and administrative officer and who not only serves at the governor's pleasure but also is subject to the orders, direction, control, and supervision of the commission members. It was found that, to a certain degree, members of the commission were involved in operational matters.

The consultant concluded that two features of the law governing highway administration should be changed: (1) Appointment terms for members of the commission should be staggered to carry over experienced members from year to year, and (2) as the commission, by law, is responsible for all matters, it, rather than the governor, should be empowered to appoint the executive director to serve at the commission's pleasure.

Although the commission is charged with the responsibility of building and maintaining the State highway system, other governmental agencis have some authority over highway affairs. The State department of administration is responsible for providing direction to other State agencies in matters concerning personnel, purchasing, and the administration of budget funds. Such control, by causing delays in giving necessary approvals and requiring cumbersome administrative procedures, has an adverse influence on the operation of the highway commission. It was the opinion of the consultant that the State department of administration should establish personnel and purchasing policy, but the execution of the policy should be the responsibility of the highway commission.

Staff or Division Organization

The term *staff*, as used in this report, applies to that level of highway organization below the chief executive, or administrator, and sometimes, the chief engineer. The different staff units are called divisions, although many State organizations use other titles such as department, bureau, group, or office for this level of management. The term *section* refers to units of management within a division.

The reasons for differences in State organizations are many; however, there is little difference among States in the basic highway functions—planning, engineering, and administration. The type and size of the management structure depends on the number and categories of personnel required to do the work, and on the expense of the highway systems.

The findings presented and discussed here do not apply to the headquarters of every State, and the recommendations for improvement in management and organization cannot be presented in an overall or general form. However, basic to each survey is a recommendation for a reorganization of the management plan not only to make it more efficient and more economical but also to meet the needs of the State being investigated.

Alabama

Principal recommendations made for reorganization in Alabama are as follows:

- Reduce the number of persons 15—reporting directly to the chief engineer.
- Centralize all accounting operations in the headquarters office.
- Expand and formalize the fiscal management, planning, and control functions.
- Establish independent audit and management systems analysis functions.

By realining and consolidating functions and organizing headquarters operations in four functional units—engineering, operations, administration, and plans and programs—the number of units reporting to the chief engineer could be reduced to six. It was recommended that each of these be headed by an assistant chief engineer—a new position and title in the management structure. This reorganization would improve coordination between staff functions; improve continuity in long-range planning and programing; relieve the highway director and chief engineer of routine administration and decisionmaking; and increase opportunities to develop top management experience and abilities.

In addition to the four coordinating units, it was suggested that two independent units—audit office and management systems analysis office—be established as staff functions of the chief engineer. The audit office's responsibilities would include independent evaluation, both financial and nonfinancial, of the degree of compliance with the department's established policies and procedures; analysis of instances of noncompliance; and special audits, as requested. Benefits resulting from such investigations would include improved compliance with policies and procedures; increased awareness of management's fiscal problems; increased reliance on financial records by other State agencies and Public Roads; and greater flexibility in adjusting to changes in operating and fiscal conditions.

The management systems analysis office would keep the department's administrative, accounting, and communications systems responsive to management needs and in conformity with management policies and statutory requirements. Benefits expected from this unit include current maintenance of all policy, administrative procedures, and procedures manuals; continual surveillance of data communications systems to keep them responsive to needs and technical advances; minimization of investment in forms; and coordination for reporting all fiscal and operational data.

To improve the fiscal and accounting function, a fiscal division as a responsibility of the assistant chief engineer, office of administration was proposed. The detailed accounting and financial recordkeeping being performed in other divisions and in field or district offices would be consolidated in an accounting section of this unit. Such an accounting and fiscal arrangement should increase the reliability of accounting records; increase the timeliness and accuracy of fiscal reports; reduce paperwork in both the headquarters divisions and in the districts; and increase the efficiency of auditing, billing, and accounting. Also proposed for the fiscal division was a financial control section to be responsible for compiling the annual operating budget, processing changes in budgets and allotments during a fiscal year, and preparing management and financial reports generated by the accounting section. Benefits expected from creation of this section are improved control over allotment of funds, improved control of district and staff functions through enforced budgeting procedures, greater flexibility in evaluating project requests for additional funds, increased knowledge of unit costs of different functions and projects, and improved reliability in long-range planning and programing.

An offices services section to consolidate headquarters offices and divisions was proposed for the office of administration. Responsibilities of this section would include office engineer services, processing construction bids, operation of a central supply room for paper stock and clerical needs, handling of all insurance and compensation claims, and operation of a centralized printing and reproduction shop. The offices services section should minimize investment in paper stocks and clerical supplies inventory, increase efficiency in procurement and field inventory operations, reduce routine forms and printing costs, and eliminate the use of unnecessary forms and records.

Georgia

In Georgia, the consultants were of the opinion that the headquarters management plan was "not arranged in a logical and functional manner." Practically all functional activities were under the direction of the chief engineer, although the statutes provide for the appointment of a director of highways and specify that he be the department's chief administrative officer. Adjustments had been made in the organization to accommodate individuals rather than functions, resulting in a sprawling structure, fragmentation of functions, and combination of unrelated activities. A plan for reorganization was submitted to limit the responsibility of the chief engineer to engineering and engineering related duties, eliminate inconsistencies, and result in more efficient highway operations.

The reorganization plan proposed organization of the department under five units or offices and included a public information office, all reporting to the director of highways. The divisions and their functions are:

Engineering—The plan specifically proposed that engineering be headed by the chief engineer and his assistants, and that it consist of seven units or divisions: Road design, bridge, construction, materials, maintenance, traffic services, and right-of-way. This arrangement is more compact and, with proper assignment of responsibility and commensurate delegation of authority, should free the State highway engineer of much unnecessary detail. Overlapping jurisdictions and responsibilities would be eliminated and the means provided for achieving better control and coordination of several units with more effective use of personnel.

Administration—It was proposed that administration be responsible for personnel, organization planning, management audits, data processing, reproduction services, files, space management, communications, and custodial services. For the data processing activities, which were being carried out at three computer installations, the consultants recommended the establishment of an adequately equipped and staffed data processing center, where the data processing services would be consolidated in one unit. The department was advised also that provisions should be made for systematic management audits of the organization and performance; that management audits should be a regular part of organization management and control; and that this activity should be placed in the administration division.

The problem of storage and retrieval of records needed attention, as record storage occupied needed working space, and necessitated an ever growing investment in filing equipment, materials, and personnel. Guidelines and suggestions were set up to systematically approach the problem. To be successful, a records management program must have the backing of top administrators, and its goals must be established, responsibility placed, and procedures outlined in a written policy statement. Responsibility for coordinating the program should be assigned to a high-ranking member of the administration division staff. The most effective way to insure compliance with a records disposal policy, the consultants advised, is to control the issuance of filing cabinets.

Finance—Units proposed for finance were financial planning, budgeting, accounting, auditing, fiscal affairs, and payroll.

Planning—The planning function had not achieved the status of a key operation within the department. It was pointed out that planning is the responsibility of the highway administrator and that a well organized and staffed planning unit to advise and assist the administrator was a necessity. It was recommended that the proposed planning division at least consist of these sections; planning—including long-range improvement plans with separate rural and urban units; programing—with separate units for implementation of cooperative projects under Federal-aid secondary and urban programs, and for developing programs in cooperation with cities and counties; and economic studies and research—including all studies and investigations, and the research needed as a basis for planning decisions.

Equipment and Supply—In the proposed management plan, the equipment and supply division would be responsible for procurement of all supplies and equipment, warehousing, supply and equipment depots and services, and radio installation and maintenance, as well as for the development and administration of an equipment rental system, self-supporting supply system, motor vehicle control, and motor pool operations. It would also be responsible for development of a training program and manual for mechanics, shopmen, and equipment operators.

Public Information Office—Proposed activities connected with public information include all relations and information services; publicity; reporting; meeting with the press, radio, and television media; annual or regular department reports and publications; and related promotion and information services. Particular emphasis was given to the publication of the annual reports and other documents, which should inform the public of the programs and goals of the department. Presentation of information in concise and simple language, it was believed, would lay the foundation for public understanding and support.

Another recommendation to insure better organization and control of the department's activities was the establishment of a standard system of organization unit designations. The use of approved titles precludes the assumption of unofficial titles and improper organization status by personnel.

The latest organization chart, indicates that the Georgia highway department has been reorganized according to some of the recommendations. The divisions of planning and personnel, and the public information office have been removed from control of the State highway engineer and made a part of the director's immediate staff. A separate utility division has been established, construction controls have been placed under the construction unit, and an internal audit section has been set up to include internal audit and systems and procedures units. Some right-of-way changes have been incorporated, and part of the reorganization of the engineering division has been adopted. Others are being studied and are expected to be adopted soon. Finance and audits was placed under the secretary and treasurer. Programing of work has been moved from the Federal-aid engineer's office to a division of planning under control of the State highway director. Few maintenance recommendations have been implemented.

Indiana

In the Indiana survey, the recommended plan, on approval of the highway commission, was implemented whenever legal enactment was not required. Implementation was at the expense of the commission.

Several operational areas of the Indiana organization were considered for improvement. There were separate units for road construction and bridge construction, each performing location, design, and other functions, and this separation was carried into the field units. Much of the design work was done by consultants, material testing facilities were limited, and traffic responsibilities overlapped other engineering functions. Some of the commission members were involved in operational activities, especially in the acquisition of land. Not only were training programs needed, but the personnel situation was critical throughout the organization, and patronage employment often was the usual procedure.

The following improvements were suggested for highway management:

- Establish an organization with definite organizational responsibilities that provide for a logical delegation of authority and group related functions under a common management.
- Develop a system to publish policies for the guidance of all supervisors in the execution of their delegated responsibilities.
- Coordinate effectively commission goals and objectives with other agencies involved in highway matters. Define commission objectives for overall plans and programs and specific objectives for each organizational unit.
- Project workloads sufficiently into the future to allow management to plan for the financing, manpower, equipment, and facilities to accomplish planned objectives. Provide managers at all levels with the information necessary to coordinate and control the operations under their supervision, and to interpret and apply policies and standards uniformly in all districts.

One of the principal objectives of the Indiana survey was to develop an organization structure and operating relationships to achieve basic goals and implement the policies of the commission. From an analysis made of the commission's responsibilities, it was recommended that highway operations be organized under five principal functional groupings, or departments, which together with their primary responsibilities are as follows:

Highway Development—Identification and design of needed highway improvements, including long-range planning, program planning, design, and related activities required to accomplish these functions at both the headquarters and district levels.

Land Acquisition—Acquisition of right-of-way to accommodate designed improvements, including abstracting, appraising, property management, and related engineering and administrative activities necessary to support land-acquisition functions.

Highway Operations—Construction and maintenance of State highways, including construction supervision, materials testing, maintenance, and traffic engineering, as well as statewide coordination, control, and execution of responsibility in the field or districts.

Technical Services—All support services used by the highway development, land acquisition, and highway operations units, including research, scheduling, computer services, photogrammetry, and office supply and maintenance.

It was recommended that the above four groups be under the direction and management of the chief engineer. The fifth group:

Staff Functions—which include personnel administration, accounting and fiscal control, management methods and procedures, and public information, was to be the responsibility of the director of highways.

Other useful tools suggested for management improvement, were guides and manuals, including:

- A position guide that defines the responsibilities of each managerial or supervisory position in the organization, and which provides a system for delegating authority to the proper levels of management.
- A management guide that contains organization charts, a position guide, and the general policies of the commission. This guide will provide a continuing means to define the functions, responsibilities, authority, and relations, of all managerial and supervisory positions.
- A functional budget that provides the manager of each major functional unit with current and accurate information regarding the financing of activities under his control, as well as a means for staffing, equipment, replacement, material requirements and other items involving the expenditure of funds.

Minnesota

In the survey of the Minnesota organization it was proposed that the State restructure its management plan around the highway department's defined objectives, and it was recommended that the organizational structure carry out the functions of the department under the following six major divisions: *Operations, planning* and *programing, engineering standards, management control, government* and *community relations*, and *safety.*

Operations—Under the supervision of a deputy commissioner, titled director of operations, this unit would have responsibility for managing the field or district units and overseeing the headquarters functions of design, right-of-way, contract administration, central shop, and road information and permits. The civil defense program and special projects were recommended to be the province of the operations division in which assignments would be headed by staff assistants. To avoid the possibility of the deputy commissioner's being too occupied with headquarters activities at the expense of the field units, it was proposed that an assistant director of operations, reporting directly to the deputy commissioner, be given responsibility for supervising the headquarters offices.

Planning and Programing—The consultants pointed out construction planning is but one of many highway-building activities and suggested that the division be expanded to include planning and programing for all the highway system's needs under the supervision of an assistant commissioner. It was proposed that the work of this division be under two principal units, transportation system planning and programing, each headed by a director. The transportation system planning section would absorb the present construction planning responsibilities of the planning research section and would include the newly assigned nonconstruction responsibilities of community transportation planning services, road system data, construction planning, nonrecurring maintenance planning, facilities and equipment planning, and programing.

Engineering Standards—In the reorganization for this division, an assistant commissioner would be responsible for assuring management that the technical operations of the department are conducted in accordance with sound engineering practice. To cover the range of technical services required, it was proposed that this new division be organized in eight units: *Research coordination*—to monitor technical research projects; *State aid*—to check the plans and specifications prepared by county and municipal jurisdictions against professional engineering standards and oversee the contract award procedure; *materials*—to provide a quality control service by conducting tests to assure that prescribed materials standards are met; *traffic engineering*—to develop standards, and review specific traffic engineering plans to assure that prescribed standards have been applied; *design standards*—to assure that road and bridge design is performed in accordance with prescribed design standards; *construction*—to develop contract specification standards, review new construction techniques, and establish contractor prequalification standards; *maintenance*—to prescribe up-to-date maintenance techniques, and to assure their use by operating personnel; and *right-of-way*—to assure top management that operating units are performing in accordance with approved practices and procedures, and to develop uniform appraisal and acquisition standards and monitor their application.

Management Control—This division would be responsible for personnel, data-processing, and accounting activities which were direct responsibilities of the State highway commissioner. In the reorganization plan, these functions would be supervised by an assistant commissioner who would be responsible for developing and using a system of management controls. The following subunits were proposed for this division: *Program control*—to monitor program progress against approved time schedules; *management services*—to provide in-house advice and guidance on organizational and management problems; *budgeting* and *financial planning*—to provide adequate budgeting services; *financial services*—to prescribe procedures and safeguards for the collection, receipt, custody, deposit, and disbursement of the department's moneys, to be responsible for all recordkeeping and manual processing of financial data, to maintain the appropriate cost accounting and control records, and to perform internal and external audits; *data processing*, to be responsible for systems and procedures improvements; and *administrative services*, to centralize the department's major housekeeping activities.

Safety—The safety function would be elevated to division status and supervision would be performed by an assistant commissioner of safety. Activities within the division would be organized under three-units, as follows: Traffic statistics analysis, highway patrol, and drivers licenses.

Government and Community Relations—This division, under the supervision of an assistant commissioner, would assume responsibility for developing a more aggressive and positive external relations effort. It was proposed that activities be administered through the following units: *Highway program information*—to prepare news releases and report on highway operations; *technical support services*—to provide specialized services in art and audiovisual techniques and to maintain the film library and publications service; and *public safety education*—to promote safety formerly in the safety unit.

Most of the consultants recommendations have been implemented in Minnesota. The department has been restructured under the six suggested major divisions, which provides the organizational flexibility to meet future needs; it has a more manageable span of control, clear cut lines of authority and responsibility, and increased delegation of authority to field districts.

The top management group has been redefined to include a commissioner, a deputy commissioner, a deputy commissioner of operations, and five assistant commissioners, and to specify their roles in program formulation and execution.

Many benefits have accrued from the external effort. Progress has been made in the areas of data processing, field maintenance, drivers license, and equipment maintenance, and has resulted in substantial cost savings. More effective approaches to government and community relations have been adopted to improve relations with the legislature and to help untangle redtape with the State department of administration.

Nebraska

The proposed reorganization for the highway department in Nebraska was formulated to facilitate adoption of the *management by objective* concept, to aline major functions and activities not considered an integral part of highway-department objectives; and to recognize differences between staff and line responsibilities.

Functions not considered necessary to the department's operations were law enforcement and public safety, scale (weighing) operations, and the keeping of accident records. It was suggested that the accident record bureau, a unit of the traffic engineering section in the existing division of engineering services and field divisions, be transferred to the department of motor vehicles, and that formal provisions be made for this agency to furnish the highway department the statistics it requires. To provide a solution to the problem presented by the other extraneous functions, it was suggested that, although legislative action might be required, a new department of public safety be established under the governor using the existing law enforcement and public safety division of the department as a basis for the new State agency. The scale operations would become a responsibility of the new agency, but responsibility for recommending to the legislature freight and dimension standards to best protect the State's road investment would remain with the department of highways.

To further restructure the activities of the department toward an integrated system of highways, roads, and streets, it was recommended that the headquarters organization consist of four major units, or offices: Administration, planning, design, and field operations. Although administrative in nature, public relations and contracts would be units under the director-State engineer because of their relation to his office. For reporting purposes legal services, under the control of the attorney general also would remain an adjunct of the office of the director-State engineer.

The four major divisions, each headed by a director, would report to the director-State engineer. These divisions and their principal functional units are as follows:[4]

Administration—Personnel, property and equipment, management information services, and data processing operations, all headed by a manager. Fiscal management under a controller.

Planning—Program planning, maps and statistics, and projects scheduling, each under a manager. Planning and research programs, county and municipal programs, each headed by an administrator.

Design—Bridge design, roadway design, materials and tests, and right-of-way, all directed by an engineer manager.

Field Operations—Maintenance, construction, and districts, all directed by an engineer.

Missouri

A regrouping of functional divisions within the highway department was recommended in Missouri to combine duplicate sections or functions, to strengthen the decisionmaking process, and to economize on time and personnel. The reorganization plan proposed that the following officials head the groups or offices indicated:[5]

- Assistant to the chief engineer, planning and design—all divisions concerned with preliminary design and planning, including right-of-way.
- Assistant to the chief engineer, operations—the divisions of construction, and materials.
- Assistant to the chief engineer, maintenance—the divisions of road maintenance, and equipment.
- Chief of administrative services, titled controller—all functions, except public information, of a staff service nature including data processing, internal review and audit, personnel, accounting, and procurement.

It was proposed that the several assistants to the chief engineer and the controller report to the assistant chief engineer. The public information function, which is involved primarily with the public relations activities of the chief engineer, should be a staff function, reporting directly to the chief engineer.

Other recommended changes in management include the establishment of a right-of-way organization and an office of district coordinator. At the time of the study, the chief counsel, in addition to his legal duties also served as chief of the right-of-way division. Assistant chief counsels handled legal matters within their areas, but decisions were the responsibility of the chief counsel. Establishment of a separate right-of-way division, and the delegation of a decisionmaking authority to the assistant chief counsels, would relieve the chief counsel of detail and permit more effective use of his services by the highway commission. Another weakness concerning right-of-way activities, and one contrary to the principle of sound fiscal control reportedly was the combination in one unit of appraisal review and negotiation assistance. It was pointed out that the establishment of property values and the negotiation with property owners should be accomplished by separate units. As time is very important in right-of-way operations, it was suggested that formal training of personnel in areas of negotiation and field liaison would reduce time spans and could result in lower acquisition costs.

The office of assistant to the chief engineer of district operations, or district coordinator, if established at the division level, would give field operations greater uniformity and more immediate supervision, and also would relieve the chief engineer of decisionmaking responsibilities when controversial issues arise between the divisions and the districts. The establishment of this office should in no way interfere with the direct lines of communication necessary for control of routine matters between these units and the management organization.

Before the recommended structural changes can be made, organizational relations and responsibilities must be defined, and specific descriptions for managerial positions prepared and approved. The assistants to the chief engineer and the controller, all recommended for appointment under the reorganized management plan, and given responsibility for coordination of the divisions assigned to them, must also be assigned adequate authority to administer the duties involved.

The following changes and suggestions were proposed for improvement of management within the highway department's headquarters divisions:

Bridge—There should be two assistant division chiefs in the bridge division—one for preliminary design, the other for final design. The maintenance section in the bridge division should be transferred to the maintenance division to make more effective use of personnel and allow the district engineer more control over the maintenance effort in his district.

Traffic—Because of overlapping functions with other divisions, the traffic division is eliminated from the revised organizational structure by transferring its responsibilities as follows: Signs and marking, to the maintenance division; sign contract, to the surveys and plans division; and traffic surveys, to the highway planning division.

Supplementary Roads—Although Public Roads regulations require that the supplementary or secondary road program be coordinated by a single unit, the level of the unit in the organization is not specified. In the consultant's opinion, the supplementary roads function could be performed and requirements adhered to if it were a section of the survey and plans division, which remove the function from division status and would not only save time, but also make more effective use of personnel and lower costs.

Personnel—The structure of the personnel division and the number of employees was considered inadequate to administer effectively the policies and programs of the highway department.

South Carolina

The management research study in South Carolina indicated that some regrouping of services and activities was necessary "in order to provide the chief executive officer with the specialized skills and knowledge needed for effective management and control over all operating divisions of the highway department."

The reorganization plan recommended that the department be organized into seven major divisions and a public relations unit. The recommended divisions are: Administration, planning and programing, supply and equipment, motor vehicle, law enforcement, engineering, and finance. It was suggested that the head of each division be called director and that some standard designations be adopted for the units of organization below the division level. It was also suggested that standardization of unit designations would aid organization control and position classification, as well as prevent unofficial acquisition of status and title. Reorganization of the department as proposed, would result in a more efficient operation because of the combination of related activities and functions into logical components. The proposed structure eliminates overlapping jurisdictions and divisions of responsibility for given functions and reduces time consuming coordination and duplication of effort now required to administer some of the activities.

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[4] The recommended headquarters organization was implemented effective January 1968.

[5] This recommendation, according to information from the Regional Office of the Bureau, is being given favorable consideration.

Administration Division—Activities recommended for inclusion in the administration division are personnel, organization planning, data processing duplication services, space management, custodial, and related housekeeping services. Because there appeared to be two independent computer operations, one engineering and the other financial, the establishment of a data processing center was recommended. When equipped with the recommended machinery and personnel, the center would provide data processing services for the entire highway department.

Planning and Programing Division—It was pointed out that planning is the key operation from which all other activities flow. The planning function, at the time of the study, was not yet developed as an organized and coordinated activity, and the responsibility for it was scattered. Planning is a primary responsibility of the chief executive, but to plan, he must have information. It is the duty of the planning division to provide this specialized assistance. It was recommended that the planning division report directly to the chief executive officer.

Supply and Equipment Division—Review of this division indicated the need for a well-defined policy on equipment replacement, or retirement, based on age, mileage, condition, and maintenance costs; proper cost records; a training program for mechanics, shopmen, and equipment operators, as well as a manual for their guidance; and the establishment of equipment operations as a self-sustaining activity with a revolving fund and proper cost-accounting and recordkeeping systems. Transfer of the purchasing function to the supply and equipment division was recommended to eliminate duplication of effort, and permit coordination of purchasing and supply and equipment management. As purchases are made by the State purchasing division, it would be the function of the supply and equipment division to supply the State purchasing division with recommendations and specifications for purchases.

Motor Vehicle Division—Inclusion of this division within the State highway department is a legal requirement. Its activities are those that deal with registration and titling of motor vehicles, revenue collection, safety and financial responsibility, and driver licensing and control.

Law Enforcement Division—This division is established by statute, and includes the enforcement of motor vehicle and traffic laws and regulations, truck size and weight, and education enforcement programs.

Engineering Division—The consultants found the engineering division to be "not arranged in either a functional or logical way." Seven district engineers, and at least a dozen other individuals report directly to the State highway engineer. Assignments are not clear, overlapping of duties occurs, and informal arrangements are the rule—all of which are signs of inadequate administrative machinery. It was recommended that the engineering division consist of seven principal sections—right-of-way, bridge, design, construction, maintenance, materials, traffic—and that it be organized to permit greater output and a better product. With proper assignment of responsibility and commensurate delegation of authority, the State highway engineer could be freed of much unnecessary detail. The combination of related functions and activities into fewer components would eliminate overlapping jurisdictions and responsibility, and provide for better control and coordination of division functions.

Public Relations Office—As organized at the time of the study, public relations and safety education were in the same unit. This unit was charged also with the administration of the school-bus driver training programs, licensing of driver training schools, and qualification and approval of their instructors.

Finance Division—The statutes require the commission to appoint a secretary-treasurer, a fiscal officer, who acts as director of the finance division. The consultants suggest that the position of secretary and treasurer be separated. Also, as the chief executive officer is the administrative officer of the department, it was suggested that he, rather than the highway commission, should appoint the director of finance and that the legal provisions concerning this officer's appointment be revised accordingly. All accounting and related functions now being carried on in other divisions should be transferred to the finance division, as should the payroll and leave records being maintained in the personnel office.

Other criticisms and suggestions for improving the department's organization and management practices were as follows:

- Management guides should be prepared to set forth the functions, responsibilities, authority, and relations of all top management positions so that occupants will have a permanent source of instruction and guidance concerning their place and functions in the organization.
- The internal auditors of the finance division should not be responsible for management audits. Reports and performance schedules should be used to check organization and performance. To assure a neutral review and appraisal, audits should be made by a division or unit not directly concerned with the operations and activities being studied. It was recommended that such a unit be placed in the proposed administration division.
- A body of policy statements, approved by the chief executive, should be developed to guide the divisions and districts in carrying out the functions of the department. Where such statements do not exist, they should be prepared for review and approval.

Since the management study was completed in South Carolina, some of the recommendations have been implemented. Some of the results affect the entire department; others affect the different divisions. Certain changes have been made in the law, although enactment may not have been influenced completely by the management study. A data processing section, placing all data processing operations of the department into one unit, with new equipment, has been established under the secretary-treasurer. In lieu of forming a planning section to work closely with the chief highway commissioner in all planning for the department as recommended, a program engineer has been installed to work with all units of the organization in planning and coordinating the different stages of construction from programing and location to acceptance of completed as part of the improved highway system.

Schoolbus driver training has been removed from public relations and transferred to the State department of education. Traffic safety education, which was also under public relations, has been transferred to the motor vehicle division. Driver license examination has been transferred from the patrol division to the motor vehicle division, and the upgrading of the ranks of patrol personnel, as suggested to improve morale, had been successfully done without materially increasing costs of operation. According to the State highway engineer, the overall review of the highway department's operations generally has improved the organization and increased efficiency.

Texas

Criticism of the highway department's structure in Texas was that it was organized according to activity rather than to function, and the organization tended to be "people-oriented" with too many people reporting to top management. Responsibilities of the divisions were divided and often overlapped; for example, five divisions, in addition to the district where work was being considered, were involved in the processing and execution of contracts. It was pointed out that this division of responsibility and duplication of activity, resulted from patterning the organization of headquarters divisions after that of the districts. The basic function of the division should be to aid management at the headquarters and district levels. A secondary function is to support the field or district organization by providing service and technical skills not obtainable within the districts. It was concluded that too much emphasis had been placed on the secondary function, which promoted independence and self-sufficiency among the units at the division level. Reorganization at the headquarters, or division, level was suggested to eliminate duplication and overlapping of functions.

Other criticisms and suggestions for improvement of the organization and management practices in Texas were as follows:

- As each division approached outside agencies independently concerning its own operations, several divisions may be negotiating with the same agency, such as the Bureau of Public Roads, on different but related matters, which has an adverse effect on planning. Moreover, the action of one division could be contrary to the best interests of another. A division's authority should be classified and specific policies put in writing so that conflicts with outside agencies can be resolved by determining which division's authority is to prevail.

- Training programs are developed and carried out by different divisions and districts without regard for the need of the department as a whole, and personnel are recruited according to the needs of the several organization units. The consultants advised that recruitment and training are proper functions of the department's personnel division. They suggested that the personnel division expand its functions to include a coordinated statewide employee training program on a continuing basis, and to develop an inventory of the skills of all personnel employed by the department. Better coordination of employment and recruitment also should be established.

Washington

In the Washington highway department, more than 20 unit heads reported directly to the director of highways. This situation was due primarily to the director's lack of authority to delegate to subordinates any of his responsibilities. Recommendations for improving and streamlining the highway department's management plan at the division level were as follows:

- Reorganize the headquarters office of the department under seven functional divisions—personnel services, management services, planning and research, engineering, construction, maintenance, and toll facilities. Such an arrangement simplifies and clarifies work flow; groups together and pinpoints authority, functional activities, and related programs; includes right-of-way and bridge design under engineering; and reduces the director's span of control.

- Redesignate or retitle major departmental positions. For the assistant director, who serves as a deputy, the title deputy director of highways could be used. The title assistant director for the heads of the proposed divisions would better depict their positions and responsibilities in the structure. Personnel director was suggested as the title for the head of personnel services. The title *assistant* applied to any position below the assistant director level should be eliminated.

- Facilitate communications between and among organizational units. The management plan, to be effective, should be flexible enough that anyone needing information can request it at the source. Subordinates should keep their superiors informed on matters affecting the operations of their units.

- Prepare functional job descriptions so that the responsibilities of each organizational unit will be generally understood. Duties and responsibilities should be set forth in writing.

- Prepare, revise, and keep operating manuals up to date.

- Establish an offices services section in the proposed management services division to provide additional office and clerical services to engineering and management personnel at all levels.

- Transfer the public information function to the office of the director of highways. In addition to duties related to public information, the public information officer should serve as the administrative assistant to the director, particularly when dealing with the public in such matters as public appearances of the director, of the members of the highway commission, and of other staff personnel.

- Strengthen the services of the legal office. Legal support is an extension of the attorney general's office. It was recommended that the legal office be headed by an assistant attorney general and that, rather than having division status in the organization, it be attached to the office of the director of highways for supervision, but report directly to the attorney general. It also was suggested that the legal staff be enlarged, and that one member of the staff be assigned to take care of legal matters in each of the district offices.

The State has two agencies concerned with toll facilities—toll bridge authority, and toll facilities division. In the opinion of the consultants, the toll bridge authority, a separate agency organized many years ago to plan and finance toll facilities, has outlived its usefulness, as most of its functions already have been transferred to the highway commission. The remaining responsibility also could be transferred without difficulty. The relationships of the toll facilities division to the other divisions of the highway department are unclear. Toll facilities are not a part of the highway department's programs and budgets, and it is difficult for them to get projects planned, designed, constructed, and maintained by the appropriate units of the department's organization. It was recommended that the toll facilities division be headed by an assistant director of highways for toll facilities who would have responsibility for coordinating, planning, and approving and monitoring design, construction, and maintenance operations performed by other highway divisions.

As of February 1968, the Washington Highway Commission had adopted and implemented, almost entirely, the recommendations of the consultants. A deputy director's position was created, and the director was delegated authority for the day-to-day management of the department. He was also given authority to subdelegate authority to subordinates. The headquarters divisions were organized into the recommended groups, and the executive heads were classified assistant directors, thus reducing the control span of the director. These changes have permitted the department to properly identify the needs for additional personnel and the qualifications required to accomplish the type of work to be done by the department.

Mississippi

A criticism of the highway-department organization in Mississippi was that no intermediate administrative-executive level existed in the organization to coordinate the work of related functions, so that all the divisions and districts independently had to report directly to the chief engineer and director. The reorganization plan submitted for this department recommended the establishment of three new management-level positions—assistant director for administration, assistant chief engineer-operations, and assistant chief engineer-planning—placed in the organizational structure between the operating divisions and the deputy director–chief engineer. The assistant director for administration was to have responsibility for:

Financial Control—On a centralized basis, including the auditing function and accounting responsibilities formerly done by the districts, the office engineer, and the materials division.

Personnel and Merit System—To centralize all personnel functions as recommended and discussed in another report; and all staff training and safety programs carried on in the department.

Office Services—A new division to administer the various incidental department-wide services formerly performed in the several divisions, and to include central files, central office supplies, mail service, printing shop, computer center, and a building services center to provide all custodial, janitorial, and space management functions for the central office.

Procurement—A new division to assume complete control of all procurement, except construction contract administration, which remains the responsibility of the office engineer.

The establishment of this new division and units in the structure will bring together all administrative functions under the assistant director for administration. It will relieve the technical divisions of miscellaneous functions thus allowing them more time for their specialized responsibilities. The improved internal control will also promote more efficient utilization of the department's resources.

The positions of assistant chief engineer-operations and assistant chief engineer-planning and design—on the same organizational level as the assistant director for administration—will not require additional personnel. The reorganization required will place the assistant chief engineer-operations in charge of staff functions that have direct relations with the field or district operations. The assistant chief engineer-planning and design will be responsible for coordination and direction of the centralized highway and design activities, including right-of-way.

These proposed management recommendations made for the division level in Mississippi have been implemented. Related functions have been regrouped under three principal offices or departments titled administration, operations, and planning and design. An assistant director heads administration, and an assistant chief engineer heads each of the other groups. Other benefits realized are improved efficiency resulting from common usage of centralized reproduction equipment; improved filing system; reduction in cost of office and clerical supplies through centralized procurement and issuing procedure; increased utilization of computer equipment in both fiscal and engineering fields, thus justifying acquisition of a more sophisticated system; and improved inventory control. Because the central offices are in five locations, the recommended services center has not been organized, but will be when new quarters become available. Also, an equipment division was established to facilitate joint use of maintenance equipment by the districts.

District Organization

Each State highway department has a field organization made up of units commonly known as districts. Highway districts are the extension of the State highway department's headquarters offices into several geographical areas of the State. It is through district organizations that highways are constructed and maintained. As there is diversity among the States in management plans at the headquarters level, there is also diversity in district organizations, and sometimes a variance within a State. Following are some of the recommendations made concerning district organization for several States.

Alabama

In the opinion of the consultants in Alabama, the existing diversity of district organization plans was deficient, primarily because of the decentralization of the department's overall organization plan. In effect, each district was a small, autonomous highway department. Differences among the districts—such as size, degree of urbanization, and topography—make differences in district responsibilities. But for overall highway operations—internal control, data flow, and general management—a basic uniform organization plan for the districts should be established. It was recommended that the operations of each district be carried out through seven major functional units—administration, right-of-way, county aid, materials and test, construction, equipment, and maintenance—all responsible to the district engineer. Benefits expected from this plan are improved management performance reporting, improved budgeting and planning, increased efficiency in administration procedures, simplification of communications and of accounting and data flow, clear lines of promotions, and improved personnel and training evaluation.

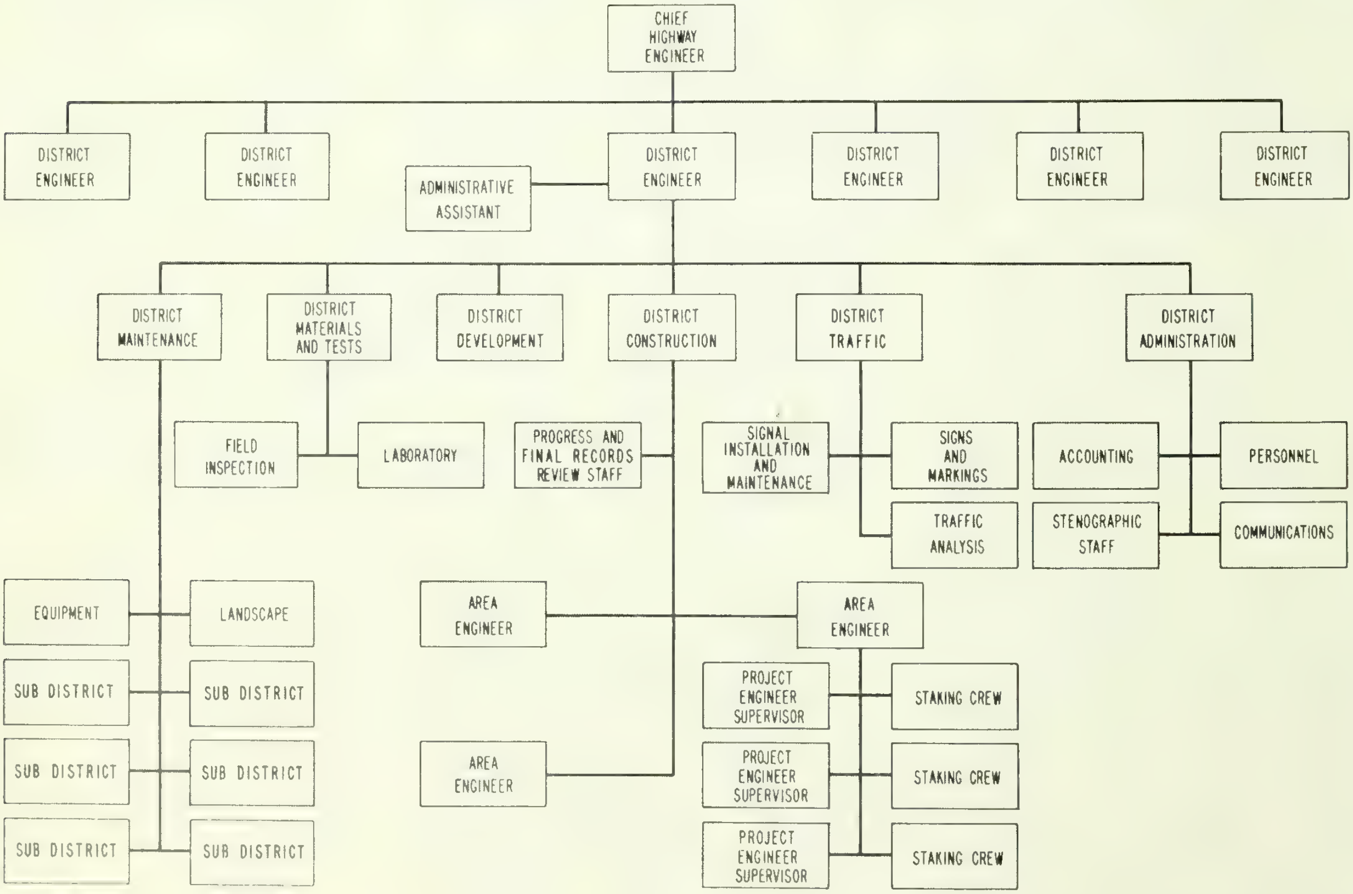
Georgia

In Georgia, where many of the highway department's activities were the responsibility of the district and subdistrict organizations, some clarification of the relations between the central office and the districts was advised. It was recommended that activities be further decentralized as qualified personnel becomes available to assume the responsibilities. It was also suggested that subdistrict organizations be eventually eliminated. Because the State is involved in the development of an integrated highway network in urban areas, it was recommended that metropolitan districts, composed of the counties making up the metropolitan areas, be established.

Indiana

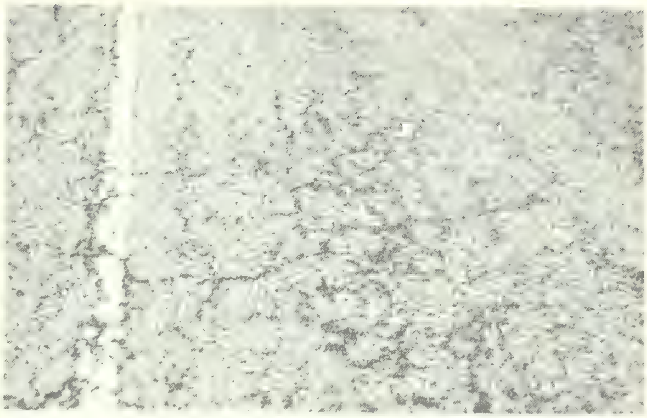
District operations in Indiana were supervised by field engineers traveling out of the central office. The highway department was advised to organize the districts following the pattern developed for the central office, and place each district under the supervision of a district engineer directly responsible to the chief engineer of the commission. The pattern provides for five functional groups—mainte-

(Continued on p. 286)



District organization used by the Indiana State Highway Commission.

It is unwise to draw conclusions from limited data when considering the durability and hardening of bituminous materials.



Asphalt Hardening—Fact and Fallacy

Reported by [1] J. YORK WELBORN,
Principal Research Engineer
Materials Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Control of asphalt hardening during construction and service of highway pavements will provide longer pavement life and result in better economy. According to the results of a study of 11- to 13-year-old pavements, the mixture design and the mixing, spreading, and compaction of the mixture at the time of construction are probably the most important factors governing the hardening of asphalt.

RESEARCH RELATED to durability and hardening of bituminous materials is documented in the literature by several hundred research reports (*1, 2*) [2]. Most of the reported research, concerned with changes in asphalts, has been conducted in carefully controlled laboratory experiments, usually under accelerated aging conditions. Well designed field experiments also have been conducted to investigate under identical environmental conditions asphalts from different sources, although usually the sources have been limited to those that were readily available in a particular area. In other research, asphalts used in normal construction under differing service and environmental conditions have been studied. Again, these studies usually were limited relatively to few asphalt sources and pavement samples.

Recognizing the need for comprehensive and reliable information, the Public Roads research staff in 1967 began a comprehensive study of asphalt-cement durability. The primary objective of this study was to determine the overall changes in the physical and chemical properties of asphalt cements during construction and service in pavements. Some of the more obvious findings concerned with the hardening characteristics of asphalt cements are reported here. The fallacy of drawing conclusions from limited data will be pointed out, and the quantitative relation between hardening and certain mixture characteristics will be shown. Some of these findings are not new but were anticipated, and it is hoped that they will serve as guidelines to evaluate asphalt durability in the continuing search for more durable pavements The complete data are now undergoing additional analyses in preparation for a later, more thorough report.

A brief description of the test procedures used in the study is included in this report. These procedures have proved suitable for studying changes in properties of asphalt and paving mixtures from pavements, and if followed by other researchers, they should provide a basis to compare data from other sources.

The research reported here is a continuation of earlier studies of properties of asphalt cements produced during 1954–56 from all major sources in the United States (*3, 4, 5, 6*). When samples of these asphalt cements were collected, specific information on the corresponding paving projects was obtained. Of the 300 projects identified 210 were constructed with 85/100 penetration grade asphalt.

In a 1965–66 survey, Public Roads obtained further information on the status of the projects, including the general condition of the pavements and the extent of maintenance or reconstruction that had been carried out since construction. This survey revealed that 130 of the 210 pavements were still in service and that the remainder had been either surface treated, overlaid, or partly reconstructed.

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[1] Presented at the 14th Annual Conference, Canadian Technical Asphalt Association, Edmonton, Alberta, November 3-5, 1969.

[2] Italic numbers in parentheses identify the references listed on page 285.

Subsequently, the condition of the pavement surfacings were evaluated to develop qualitative and quantitative information on the types of distress observed. For this phase, the hypothesis was that the type and amount of distress may be correlated with the original or existing asphalt properties, or with the changes in the properties during service. A report on the procedures used for the pavement evaluation was presented at the 1969 meeting of the Association of Asphalt Paving Technologists and will be published in the Proceedings of AAPT, vol. 38, 1969.

Field Sampling Plan

To reduce the amount of work required to obtain and test samples from all the 210 paving projects, a representative group of pavements was selected for study. The selection was based on the information from the 1965–66 survey and on several other factors related to the asphalts—their physical and chemical properties, their sources, and the environments at the locations where they were used. Considering that overlaid or surface-treated pavements could indicate asphalts of poor quality, a number of *nonsurvivor* projects were included. Fifty-three projects in 19 States were finally selected for detailed study—38 were *survivors*, or in service as constructed, and 15 had been resurfaced. The projects represented asphalts from 29 different sources. Seventeen of the 29 asphalts had been used in more than one paving project and eight were represented by both survivor and nonsurvivor pavements. To provide data on one major source of asphalt, three projects were included that had been constructed with 60/70 penetration asphalt. The locations of the projects by States are shown in figure 1.

A random sampling plan was used for each of the paving projects. Initially the plan called for six samples per project, but as the field study progressed, the frequency of sampling was sometimes modified, primarily for projects that were less than 1 mile long. The minimum number of samples taken for any project was two; the total number of samples taken from the 53 projects was 313.

During the pavement-condition evaluation the survey team located each sample site and marked it with spray paint. The section of pavement 200 feet beyond the sample site was used for the condition evaluation.

Sample locations were in the outer wheel path of the travel lane, an important feature of the sampling plan. For a special study of transverse variability, two projects also were sampled between the wheel paths and in the passing lane. Confining the sample location to the outer wheel path tended not only to eliminate differences owing to construction compaction and traffic densification, but also to minimize the effect of oil drippings.

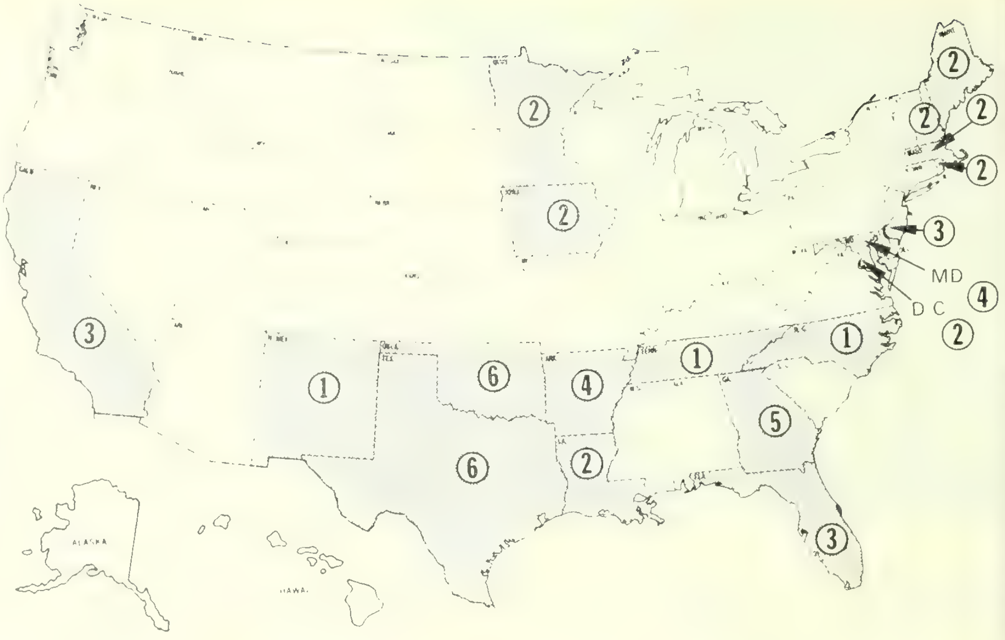

Figure 1.—States included in study.

Laboratory Analysis

All pavement samples from the 53 projects were processed and analyzed in the Public Roads research laboratories. Prior to the laboratory analysis, a preliminary study was conducted to determine the adequacy and precision of all the laboratory tests to be used. Four replicate tests were made on split samples from four pavement samples, and the variability of results determined. This preliminary study showed that all procedures, general descriptions of which are given later under *Laboratory Test Procedures*, were under satisfactory control.

In addition to the laboratory study of *physical* properties of the paving mixtures and asphalts, the *chemical* composition and changes in composition caused by aging were investigated by a contractor who had previously studied the composition of the asphalts representative of those used in the pavements (*5*, *6*).

As test data were accumulated, it became evident that there was a wide variability in results of samples from the same project. Examples of the variability are shown in table 1 for six pavements in which the same asphalt was used. The test properties tabulated for recovered asphalts are 77° F. penetration, and 140° F. viscosity; those for the paving mixtures are air void content and voids filled with asphalt. Variability values shown in the table for six samples are minimum, maximum, mean, and standard deviation. The largest standard deviations for these six projects were 17.6 decimillimeters for penetration, 104 kilopoises for viscosity,

Table 1.—Variability of test data for properties of pavement samples

Property	Project number	Variability			
		Minimum	Maximum	Mean	Standard deviation
Penetration, 77° F	17	22	50	31. 5	9. 6
	18	31	70	47. 6	14. 4
	26	18	24	21. 3	2. 3
	27	23	74	46. 7	17. 6
	40	29	39	35. 0	4. 3
	41	17	38	23. 8	7. 3
Viscosity, 140° F . . . kilopoises	17	3. 9	36. 7	19. 0	12. 7
	18	2. 5	17. 0	9. 1	5. 6
	26	35. 5	60. 0	46. 8	7. 9
	27	2. 1	32. 0	11. 6	11. 2
	40	7. 0	12. 7	9. 4	2. 4
	41	13. 4	315. 2	119. 0	104. 0
Air void content . . . percent	17	0. 1	4. 9	2. 5	1. 5
	18	1. 0	2. 9	1. 8	0. 8
	26	4. 4	7. 2	5. 7	0. 9
	27	1. 0	5. 3	2. 3	1. 5
	40	2. 1	2. 9	2. 4	0. 4
	41	2. 7	12. 1	6. 9	3. 0
Voids filled with asphalt . . . percent	17	75. 4	99. 5	86. 8	7. 5
	18	84. 1	94. 0	89. 4	4. 3
	26	64. 3	75. 7	70. 0	3. 7
	27	73. 4	93. 3	87. 4	6. 8
	40	82. 5	87. 5	85. 6	2. 2
	41	47. 2	85. 0	67. 5	12. 1

3.0 percent for voids, and 12.1 percent for voids filled. Inspection of these data, as well as of data for individual samples, indicated that differences in the properties of recovered asphalts were due to differences in the properties of the mixtures, as will be shown.

Effect of Voids on Asphalt Hardening

The relation between void content and penetration of asphalt No. 7 after 12 years of service is shown in figure 2. The plotted points are for 30 individual samples of the six projects tabulated in table 1. Some scatter of results is evident, but there is a definite trend of lower penetrations with higher void contents, as might be expected.

The relation can be described by a least squares regression line of the form log penetration, $Y=1.783-0.599$ log voids, X. Four other lines, parallel to the regression line, are also shown in figure 2. The two dashed lines show the limits of plus and minus *one* standard error of Y on X. The other two lines, shown by dash–dots, are the limits of plus and minus *two* standard errors of Y on X about the line of regression. Should asphalt No. 7 be sampled and tested under a similar program, 68 percent of the values for penetration and voids would be expected to fall within the one-standard-error band, and 95 percent within the two-standard-error band.

The relation of pavement-sample void content to asphalt hardening, as indicated by viscosity at 140° F., is shown in figure 3. The relation is similar to that shown in figure 2. It is apparent from these figures that several samples are necessary to reveal the range in the data when hardening of asphalt in pavements is studied. Many reports on asphalt hardening during service have data based only on one sample location. Moreover, some reports compare the hardening of asphalts from different sources without recognizing the relation of mixture properties. For example, asphalt No. 7 as used in State C could be considered a nondurable asphalt, as the low penetration values, below 30, usually are associated with pavement distress, depending on the climatic environment. However, many of the penetration values for the same asphalt from projects in States A and B were above 30, and several were more than 50. The higher penetrations have been associated with good performance under the same environmental condition.

Accordingly, to judge asphalt durability from insufficient data is fallacious. If variability of the paving mixture throughout the pavement is not known, several random samples must be tested. Also, if significant variability is found, the properties of the recovered asphalt must be related to the mixture properties before the durability of the asphalt is judged.

Voids Filled With Asphalt and Hardening

Other mixture properties associated with performance of asphalt No. 7 are shown in figures 4 and 5. The relation of voids filled with asphalt to penetration of the recovered asphalt after about 12 years of service is shown in figure 4. As might be expected, the relation, an *eye-ball* curve, indicates a definite trend of less hardening for higher percentages of voids filled. For this particular asphalt, from the standpoint of hardening, a minimum of 85 percent voids filled probably would have provided relatively good performance for all projects.

The relation of voids filled to hardening, as measured by viscosity at 140° F., is shown in figure 5. This curve, based on a least squares computation, indicates that the relation is linear for this type of plot–log viscosity against voids filled. Also shown in figure 5 are the limits established by computations of one and two standard errors. The difference in the shapes of the curves in figures 4 and 5 could be due to the fact that consistency of hard asphalts can be measured better by viscosity at 140° F. than by penetration at 77° F. Also, it is possible that the complex flow, or shear susceptibility, of these asphalts at 77° F. interferes with the true measurement of consistency. However, regardless of the method of measurement, the important role that mixture design plays in the durability of the contained asphalt is illustrated by figures 2–5.

The discussion heretofore was based on the analysis of only one of the 29 asphalt sources studied. Within-project data from other asphalt sources show similar relations between mixture properties and asphalt durability. The extent of within-project variability varied from project to project but is not included in this report. The relations of void content and voids filled with asphalt of the mixtures with penetration and viscosity of the asphalts recovered from all the pavements are shown in figures 6, 7, and 8.

The relation of void content to penetration of recovered asphalt is shown in figure 6. Plotted points are average values for each of the 53 projects. Although the plot points are

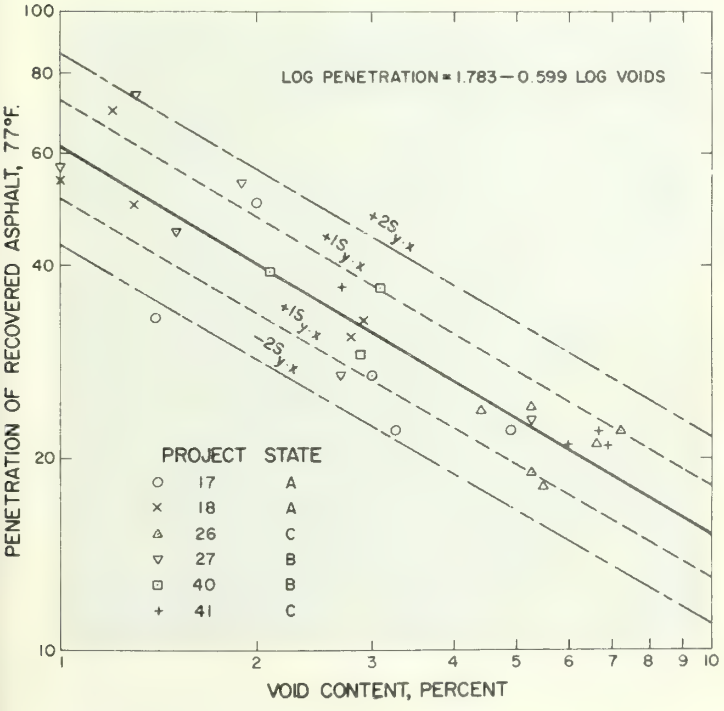

Figure 2.—Effect of void content on penetration of recovered asphalt—AC-7.

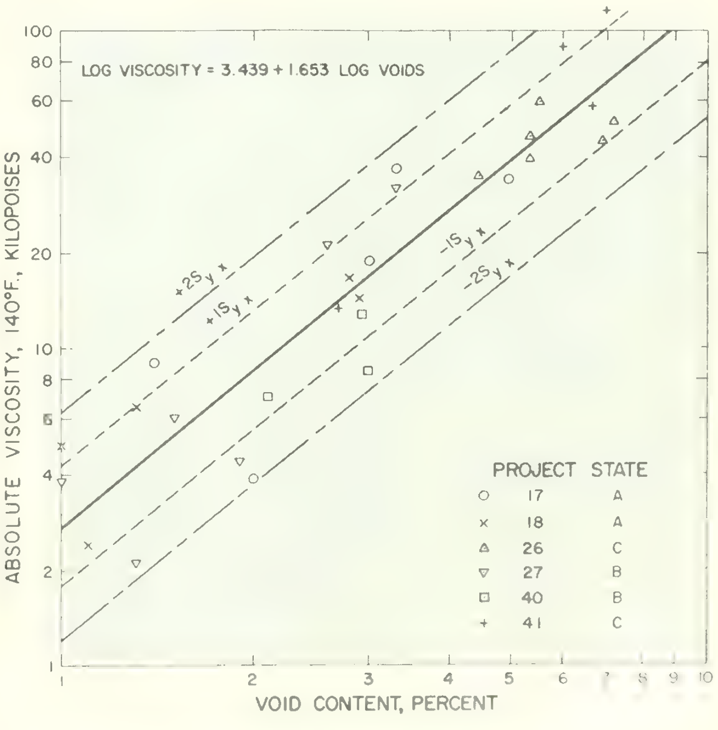

Figure 3.—Effect of void content on viscosity of recovered asphalt—AC-7.

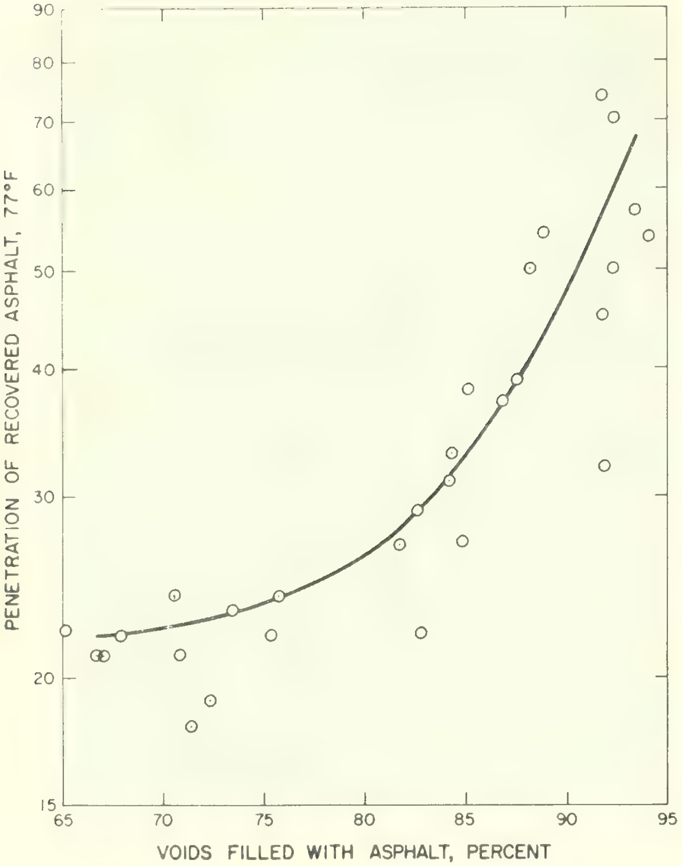

Figure 4.—Effect of voids filled with asphalt on penetration of recovered asphalt—AC-7.

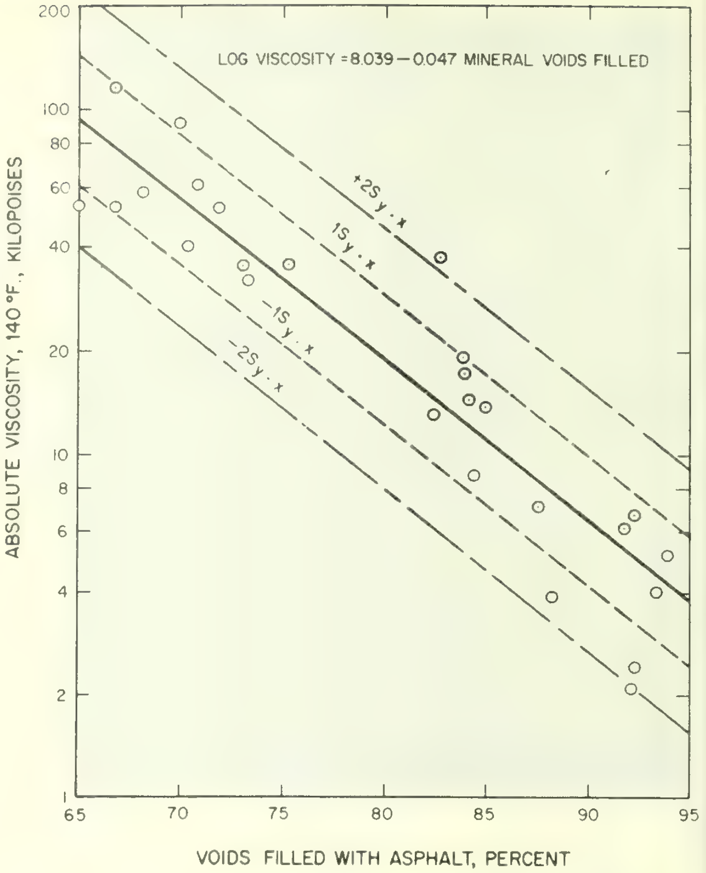

Figure 5.—Effect of voids filled with asphalt on viscosity of recovered asphalt—AC-7.

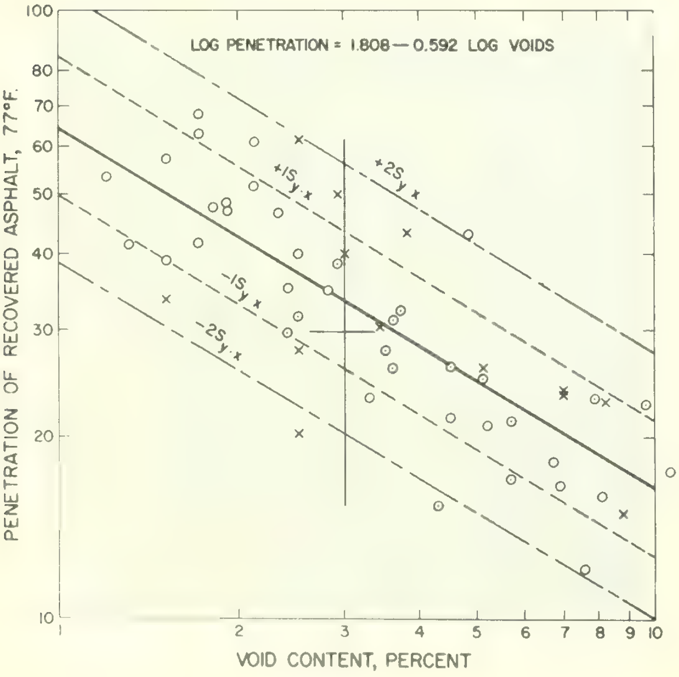

Figure 6.—Effect of void content on penetration of recovered asphalt—averages of projects.

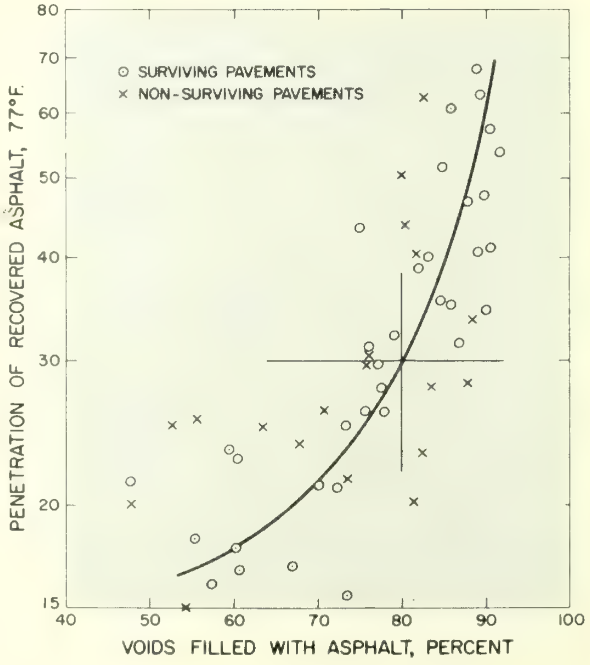

Figure 7.—Effect of voids filled with asphalt on penetration of recovered asphalt—averages of projects.

scattered appreciably, the plot indicates that pavements with no more than 3-percent voids have acceptable levels of hardening. Only three projects having voids of less than 3 percent had penetrations of less than 30. For five projects, the void contents were more than 3 percent and penetrations were above 30.

The relation of voids filled to penetration of recovered asphalt is shown in figure 7. Based on the trend shown by the arbitrarily drawn curve, only four projects with more than 80-percent voids filled had penetrations of less than 30.

The relation of voids filled to viscosity at 140° F. of the recovered asphalt is shown in figure 8. The asphalts from all but two of the projects had viscosities of less than 30 kilopoises when the voids filled were above 80 percent. Only four projects with 70 to 80 percent of the voids filled had asphalt viscosities of less than 30 kilopoises. Moreover, the spread in the results of figures 6, 7, and 8 indicate that factors other than void content and voids filled affected the hardening of the asphalt.

To determine the differences in the durability characteristics of the various asphalts, these data, together with data on such mixture properties as volume of asphalt, mineral voids, surface area, and type of aggregate should be analyzed to determine their contributions to changes in the physical and chemical properties of the asphalts.

Viscosity data were obtained at low temperatures. Eventually, these data also should be analyzed to obtain information on temperature and shear susceptibility of the recovered asphalts and on their changes during service. Such information should determine whether viscosity measurements over a range of temperature can be used more advantageously than conventional penetration, softening point, and ductility tests to analyze performance characteristics of asphalts.

As indicated previously, the following test procedures were included in this report because they have proved suitable for studying the properties of paving mixtures and recovered asphalts. They should provide a guide to other researchers considering similar studies.

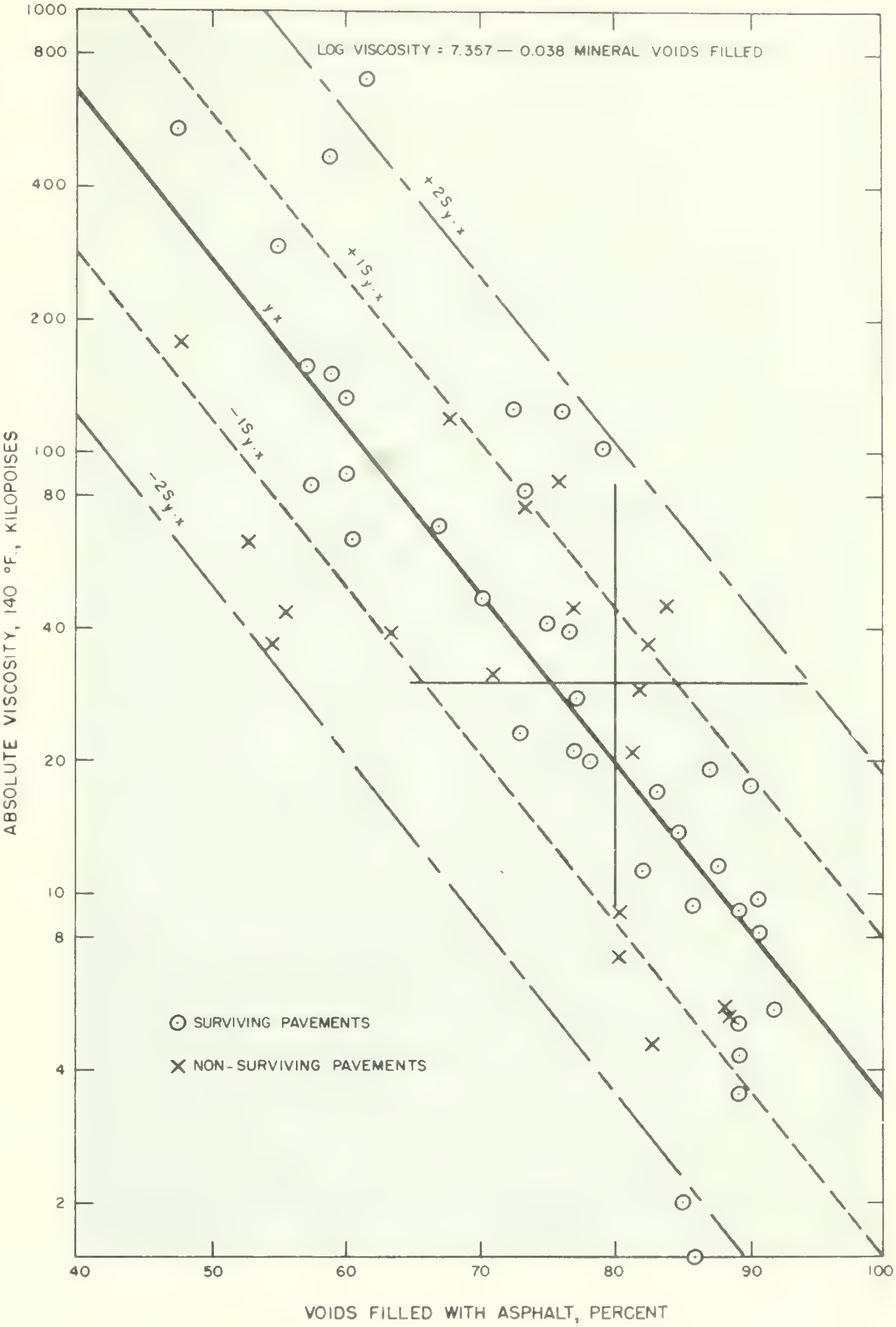

Figure 8.—Effect of voids filled with asphalt on viscosity—averages of projects.

Laboratory Test Procedures

Following is a description of the procedures used to determine the physical characteristics of pavement samples, bituminous mixtures, and recovered asphalts. Each test was assigned to the same operator throughout the testing program, thereby eliminating test-result differences caused by individual testing techniques.

Receipt of pavement samples

All samples received from the State highway departments were properly packed; no samples were distorted or broken. Samples were properly identified according to instructions given to the highway departments.

Samples received at the Public Roads research laboratory were logged in, the various pavement layers measured, and Polaroid pictures taken to record the type of sample submitted. With few exceptions, the samples contained the underlaying asphalt binder or base courses.

Sample preparation

To insure removal of surface contaminants, the top ¼ inch was removed with a diamond saw. If the test layer had been resurfaced, the overlay plus ¼ inch of the original surface was removed. With few exceptions, 1 inch of the original surface course from each core or slab was selected for test. The material above and below the 1-inch layer were stored in the laboratory for future study. Usually, enough pavement samples were processed at one time to permit about four analyses during each working day.

Bulk specific gravity

On each test sample, bulk specific gravity was determined by the water immersion methods described in ASTM D 1075–54 (AASHO T 165–55). The volume was determined as the difference between the saturated surface-dry weight and the weight of the test sample in water. After immersion in water,

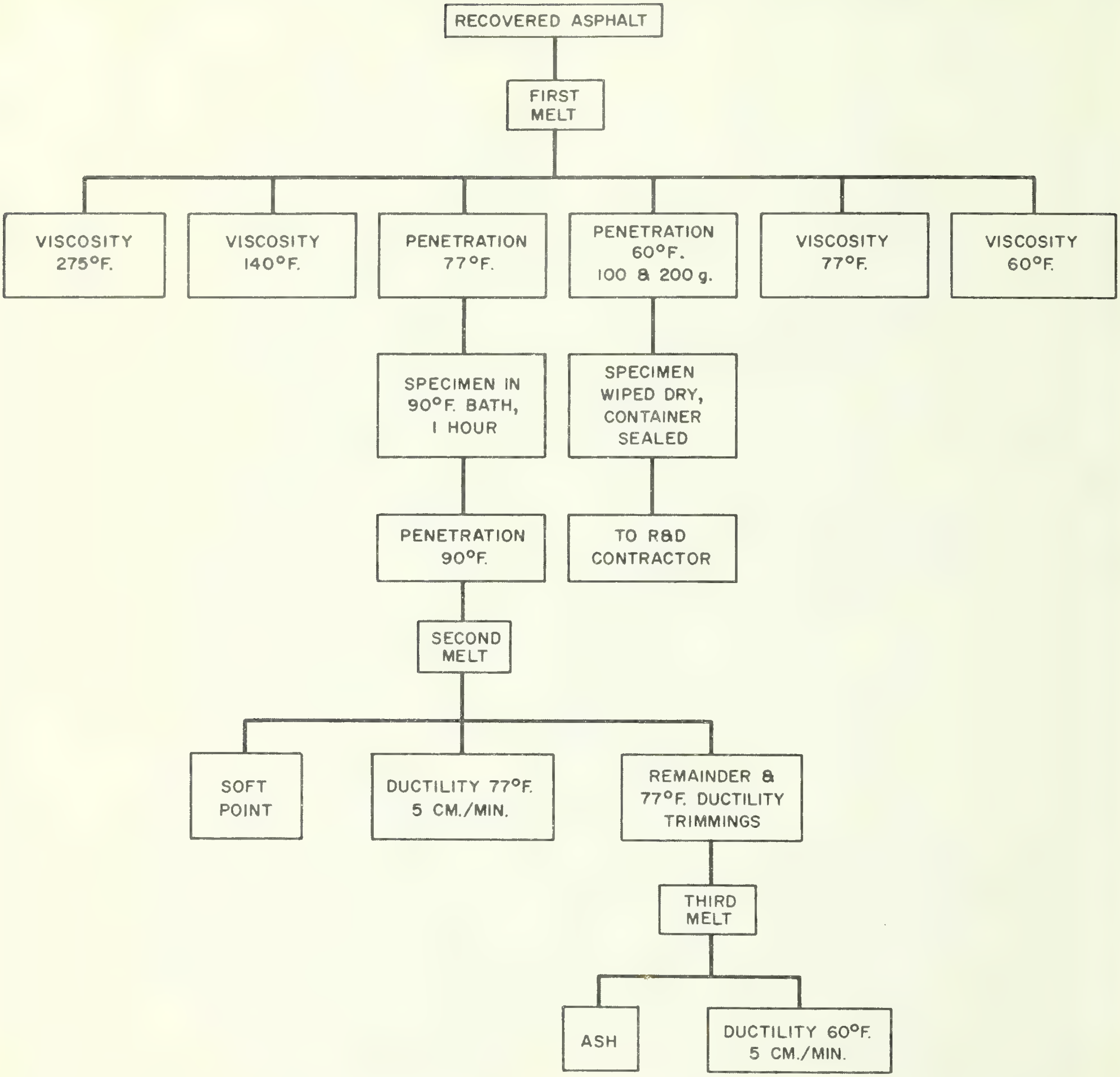
RECOVERED ASPHALT
FIRST MELT
VISCOSITY 275°F.
VISCOSITY 140°F.
PENETRATION 77°F.
PENETRATION 60°F. 100 & 200 g.
VISCOSITY 77°F.
VISCOSITY 60°F.
SPECIMEN IN 90°F. BATH, 1 HOUR
SPECIMEN WIPED DRY, CONTAINER SEALED
PENETRATION 90°F.
TO R&D CONTRACTOR
SECOND MELT
SOFT POINT
DUCTILITY 77°F. 5 CM./MIN.
REMAINDER & 77°F. DUCTILITY TRIMMINGS
THIRD MELT
ASH
DUCTILITY 60°F. 5 CM./MIN.

Figure 9.—Testing schedule for recovered asphalts.

the samples were dried for 3 hours in a vacuum oven at 29.8 in. Hg and 115°–120° F. This procedure insured a dry sample with no entrapped water and no additional hardening of asphalt. After cooling, the dry weight of the test sample was determined and the bulk specific gravity calculated.

Enough dried cores from each sample were randomly selected to provide 3,200–4,000 grams of mixture. The cores were placed in a pan and heated by three 250-watt infrared lamps to a softness that permitted the particles to be separated. After hand mixing in the pan, the *broken down* samples were split by riffle sample splitter to obtain test samples of 2,000–3,300 grams for extraction and recovery and of about 1,200 grams for the maximum specific gravity test.

Maximum specific gravity

The maximum specific gravity of the mixture was determined by the vacuum saturation procedure according to the method outlined in ASTM D 2041–64T, (AASHO T 209–64). A wetting agent was used in the water to facilitate complete wetting of test specimens. Using maximum specific gravity, the average bulk specific gravity of the test specimens and the percentage of asphalt in the mixture were calculated for samples from each site—percentage of air voids, percentage of mineral voids (based on the effective specific gravity of the aggregate), and effective specific gravity of the aggregate.

Sieve analysis

Dried aggregate from the extraction operation was used for sieve analysis, AASHO T 30–55. The aggregate sample, representing combined material from one site, was washed to loosen material adhering to the larger size aggregate and to remove most of the material passing the 200 sieve by decantation over a No. 200 sieve protected by a 40-mesh sieve. A wetting agent was used in the water. The aggregate sample was ovendried and then separated on the 1-inch, ¾-inch, ½-inch, and ⅜-inch sieves by handshaking, and on the No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200 sieves by mechanical shaking. The sieve analysis included the amount of fines removed by washing and the ash contained in the recovered asphalt. In addition to the sieve analysis, the surface area of the aggregate was determined using the following formula:

Surface area (ft.²/lb.) $=2+0.02a+0.04b+0.08c+0.14d+0.30e+0.60f+1.60g$

Where,

a, b, c, d, e, f, and g are percentages of total aggregate passing sieve sizes Nos. 4, 8, 16, 30, 50, 100, 200, respectively. The surface area (S.A.) was corrected to a common basis of specific gravity of aggregate (2.65):

$$\text{Correct S.A.}=\frac{\textit{Effective Specific Gravity}}{2.65}\times\text{calculated S.A.}$$

Additional data

The following additional mixture characteristics were calculated from determined values:

- Bitumen index $\times 10^{-3}=\dfrac{\%\text{ asphalt, (aggregate basis)}}{100\times\text{corrected surface area}}$
- Voids-bitumen index ratio$=\dfrac{\%\text{ air voids}}{\text{bitumen index}\times 10^{-3}}$
- Dust/A.C. ratio$=\dfrac{\%\text{ minus No. 200 sieve}}{\%\text{ A. C., (aggregate basis)}}$
- Film thickness = bitumen index $\times 10^{-3}\times 4.870$

Where, 4.870 is a factor developed by Campen and others (7).

- Percent mineral voids.
- Hudson $\bar{A}$ value.
- Ratio of aggregate passing No. 8 and No. 200 sieves.
- Ratio of aggregate passing No. 50 and No. 200 sieves.

A petrographic analysis of the coarse and fine fractions of the extracted aggregate was made to determine the classification and type of aggregate used on each project.

Extraction procedure

The test samples obtained as described above were extracted in single 2,000–3,000-gram-capacity centrifuge bowls in accordance with AASHO T 164–65, Method *A* using A.C.S. grade benzene. Because the samples had been vacuum-dried, water determinations were not made.

Immediately after the extraction was completed, 100-ml. aliquot was taken for an ash correction; the asphalt was recovered from the extract by the procedure that follows.

Recovery procedure

The extracts were clarified in a continuous-flow, super centrifuge at 48,000–50,000 r.p.m. and concentrated to about 200 ml. by vacuum distillation under 25–27 in. Hg of vacuum. Check tests of this procedure indicated that the maximum temperature of the residue in the distillation flask was about 180° F.

The concentrates were transferred to 250-ml. wide-mouth extraction flasks and the asphalts recovered in accordance with the procedure described in ASTM D 1856–65, (AASHO T 170–65). After completing the final distillation, the recovered asphalts were poured into 6-ounce ointment-type cans for testing. Generally, the amount of asphalt obtained in each recovery test was of the order of 70–90 grams, depending on the composition of the mixtures. On the average, about 7 hours were required for four complete extraction and recovery tests.

Tests on recovered asphalts

When available, standard procedures were used for tests on the recovered asphalts. The sequence of the tests and the general handling of the test samples is shown in figure 9.

The following specific procedures were used for the tests:

Viscosity

275° F—ASTM D 2170–65T (AASHO T 201–65) (Zeitfuchs).

140° F [1]—ASTM D 2171–63T (AASHO T 202–65) (Cannon Manning).

140° F [2]—ASTM 1968 Book of Standards, part 11, pp. 864–865, sections 6(a) and 6(b).

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[1] For viscosities up to 200 kilopoises)

[2] (For viscosities 200 kilopoises—1 megapoise).

77 and 60° F—ASTM 1968 Book of Standards, part 11, pp. 839–844, (Controlled Shear Rate Method).

Penetration

ASTM D 5–61 (AASHO T 49–64):

60° F., 77° F., 90° F.,[1] 100g., 5 seconds.

60° F., 200g., 60 seconds.

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[1] 90° F. test made on 77° F. test specimen after 1 additional hour of conditioning in 90° F. bath.

Softening point

ASTM D 36–64T (AASHO T 53–64), except that a four-specimen ring holder was used.

Ductility

ASTM D 113–44 (AASHO T 51–67I), except that one test specimen was used and no adjustment of bath specific gravity was made.

Ash

AASHO T 111–42, except that residue was not recarbonated, as ignition was controlled at 550–600° C.

Summary

Some of the more obvious findings of a comprehensive study of the durability of asphalt cements subjected to 11 to 13 years of service in pavements has been presented here. The data serve to illustrate the fallacy of judging durability of asphalts from limited pavement samples and without knowledge of mixture properties. The data also confirm or establish the following facts:

- Void contents of asphalt concrete paving mixtures are related to durability of the contained asphalts. Reducing voids to less than 3 percent assures improved asphalt durability during service. However, excessive reductions that promote instability and flushing under traffic should be avoided.
- Voids filled with asphalt have a definite relation to asphalt durability. At least 80-percent voids filled assures more durable asphalt. As for void content, excessive filling should be avoided.
- Factors other than void content and voids filled with asphalt contribute to the lack of asphalt durability.

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(7) *The Relationships Between Voids, Surface Area, Film Thickness and Stability in Bituminous Paving Mixtures*, by W. H. Campen, J. R. Smith, L. G. Erickson, and L. R. Mertz, Proceedings of the Association of Asphalt Paving Technologists, vol. 28, pp. 149–178, January 1959.

Fatal Head-On Collisions on The Interstate System, 1968, Caused by Wrong-Way Drivers

Harold R. Hosea, Office of Traffic Operations, Bureau of Public Roads

In studies of fatal accidents on completed sections of the Interstate System in 1967 and 1968,[1] it was disclosed that approximately 5 percent of the crashes were head-on collisions caused by drivers operating their vehicles in the wrong direction. Although not a large proportion of the accidents that occur, these crashes are especially important for two reasons. First, in 1968 for example, they resulted in a higher fatality rate per accident than any other type of collision. The estimated economic loss in fatalities, personal injuries, and property damage for the 1968 wrong-way-driver collisions was $9.5 million (computed as described on page 219 of the original article [1]). Secondly, it was anticipated that the design of the Interstate System, with its fully controlled access and separated opposing lanes, would minimize head-on collisions.

Following an investigation of wrong-way movements on fully controlled access facilities in 1964, a circular memorandum from the Bureau of Public Roads to its regional and division engineers stated this conclusion: ". . . it was hoped that most of the wrong-way movements would tend to solve themselves with time, yet this has not proven to be the case." At the time that this investigation was conducted, 15 percent of the States reported that the problem of wrong-way movements was serious enough to warrant extensive studies to determine remedial steps, particularly with respect to signing.

As suggested in the 1968 study of fatal accidents,[1] the problem continues to be serious, despite improvements in signing and other engineering features. For example, in 1968, 35 States reported at least one fatal accident of the wrong-way type and two reported 16 each. The 1968 study also confirmed a conclusion based on the 1964 investigation that older drivers account for a disproportionate number of wrong-way movements. The median age of drivers responsible for the 1968 wrong-way fatal collisions was approximately 39, compared with a median age of 32 for all fatal accidents on the Interstate System. Drivers 65 years of age and older were responsible for 15.3 percent of the 1968 crashes caused by wrong-way driving; only 5.4 percent of the drivers responsible for all fatal accidents were in this age group.

At least two circumstances seem responsible for the preponderance of older drivers involved in wrong-way accidents. Presumably there tends to be a decrease in alertness with advancing age. More significantly perhaps, many older drivers acquired their driving habits prior to the widespread development of freeways and, therefore, may be more prone to make U-turns on a divided highway if they find that they are traveling in the wrong direction or have missed the desired off-ramp. Such maneuvers contribute significantly to wrong-way accidents, although entering freeways by exit ramps is probably the more common violation.

An important factor not considered in the 1964 investigation is the drinking driver. In the 1968 study it was found that 32 percent of the drivers primarily responsible for all fatal accidents whose sobriety was reported had been drinking and that 9 percent were obviously intoxicated. The corresponding percentages for drivers responsible for wrong-way fatal accidents were 74 and 33 percent. The wrong-way drivers reported as drinking were also generally older than those involved in other types of fatal accidents.

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[1] *Fatal Accidents on Completed Sections of the Interstate System, 1968*, PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, vol. 35, No. 10, October 1969, pp. 217-224.

State Highway Department Management

(Continued from p. 278)

nance, materials and tests, development, construction, and traffic.

Minnesota

It was recommended that each district engineer in Minnesota be designated chief executive responsible for implementing the highway program in his district: He should have complete freedom of action as long as he operates within the limits of the approved program plans given him for execution. Organization within a district should depend on the size of the district, permitting each supervisor a span of control to enable him to provide adequate supervision of his subordinates. Structuring the plan of organization for the district in accordance with the volume of their engineering and construction work has been done. Benefits are equalization of workloads, improved morale, improved performance in field decisions, increased expertise at district level due to specialization, and establishment of additional career ladders.

Missouri

Each district in Missouri is supervised by a district engineer who can be considered the field counterpart of the chief engineer of the State highway department. Because the district engineer must devote his time and attention to the general public, as well as to internal operating problems, there should be an assistant district engineer in each district.

South Carolina

The system by which the districts are related to the highway department in South Carolina is described as decentralized administration with centralized controls. To strengthen this arrangement, and free the district engineers of routine matters, thus permitting them to give attention to the general planning, management, and public relations responsibilities, the following personnel assignments in the district organization were proposed: An office manager responsible for accounting, financial records, payrolls, time and attendance, purchasing, inventory and related office services; a traffic engineer, responsible for the normal traffic engineering activities, particularly in the urban districts; and a materials engineer, reporting to the district construction engineer, responsible for the district laboratory and for testing and checking materials.

Texas

The diversity of management plans within a State, even when workloads are of similar proportions, is shown by this finding. In Texas for example, the district laboratory in one district is under the district design engineer, in another the assistant district engineer who also supervises accounting and warehousing, whereas the latter functions in the first district cited are assigned to the district maintenance engineer. Such organization patterns tend to emphasize differences between districts similar in other respects, and detract from the effective coordination of highway activities by the headquarters offices. It was suggested that appointments be made to fill given positions rather than shifting functions to fit the ability of the incumbent.

Washington

Relations between the districts in Washington and the headquarters divisions should be clarified. As the districts are not autonomous units, headquarters should establish standards for district operations and the district should be required to operate within the standards

Conclusions

Highway management personnel will need to keep aware of changing State requirements and expectations of highway users. They must be prepared to manage a larger and ever-expanding program. Highway departments need to have a plan of organization matched to these rapidly changing and developing programs. The changing requirements and expectations must be kept under continuous review in order to provide goals that meet them, provide a source of directed action for the highway department, and provide a cohesive team approach on the part of all elements of geographically widespread departments.

Some States may not require all the organizational units recommended in the States surveyed—or perhaps cannot afford them. However, many of the problems arising in the larger highway organizations also arise in the smaller ones and the recommendations and findings reported may offer a solution when modified to the situation.

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New Publications

Five new publications by the Bureau of Public Roads may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Reference Guide Outline—Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways

Reference Guide Outline—Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, revised 1968 ($1.25 a copy), enables the purchaser and the producer of aerial photography, maps, and other survey data required for highway engineering to know what is needed and what can be furnished to effectively and economically obtain all essential data. This revised Outline incorporates many helpful suggestions from users of previous editions, including, various highway engineering organizations and photogrammetric engineering firms.

The National System of Interstate and Defense Highways

The National System of Interstate and Defense Highways, 1969 (15 cents a copy), is a pamphlet with illustrations and a map about the 42,500-mile Interstate and Defense Highway System which, when completed, will connect and serve 90 percent of all cities of over 50,000 population, as well as many smaller cities and towns. The map shows highways open to traffic, major toll roads, and plans for roads not yet under construction.

Standard Plans for Highway Bridges, Vol. III, Timber Bridges

Standard Plans for Highway Bridges, Vol. III, Timber Bridges, May 1969 (75 cents a copy), is a revised edition of the 1962 publication. The plans are intended to serve as a useful guide to State, county, and local highway departments in the development of suitable and economical bridge designs, and to give sufficiently complete information so that they will approach contract drawings as nearly as practicable. For any given bridge location, however, the requirements imposed by site conditions will necessitate modification of the plans, which have been developed for level roadway grades and right-angle crossings only.

Presented in this volume are five series of standard plans for timber bridges as follows: Timber stringers—laminated deck, timber stringers—concrete deck, continuous longitudinal laminated deck, continuous timber-concrete—composite deck, and glued-laminated stringers—laminated deck.

Analysis and Modeling of Relationships Between Accidents and the Geometric and Traffic Characteristics of the Interstate System

Analysis and Modeling of Relationships Between Accidents and the Geometric and Traffic Characteristics of the Interstate System, August 1969 ($1 a copy), presents the results of a research effort to relate the geometric and traffic characteristics of the Interstate System to the accident experience of the System. Mathematical models were developed for 13 individual unit types, such as outer connections, loops, and acceleration lanes, and six interchange types, such as diamond, cloverleaf, and trumpet. These models provide a measure of the relation between accidents on Interstate highways and the various geometric and traffic characteristics.

Geometric characteristics studied in this project had very little bearing on the frequency of accidents on the Interstate System. Although there was no warrant for any major changes in geometrics, there was evidence that minor changes in geometrics would effect some change in the accident rate.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, 1969 ($3.50 a copy), is patterned largely after the AASHO Guide Specifications for Highway Construction to establish nationwide uniformity and consistency of specifications and contractual matters in highway construction. These specifications are designed to prevent early obsolescence of the specifications and controversy in the administration of contracts, and to encourage increased production and decreased costs consistent with high quality work.

This publication is issued primarily for use in the construction of roads and bridges on Federal highway projects under the direct supervision of the Bureau of Public Roads.

Although the specifications are for those items of work, materials, and construction methods that are generally applicable to direct Federal highway contracts, the information is adaptable for use by other highway agencies,

Highway Research and Development Reports Available From Clearinghouse for Federal Scientific and Technical Information

The following highway research and development reports are available from the Clearinghouse for Federal Scientific and Technical Information, Sills Building, 5285 Port Royal Road, Springfield, Va. 22151. Paper copies are priced at $3 each and microfiche copies at 60 cents each. To order, send the stock number of each report desired and a check or money order to the Clearinghouse. Prepayment is required.

Highway research and development reports available from the Clearinghouse are also listed by subject in Public Roads annual publication Highway Research and Development Studies *(see inside back cover) according to the goals and projects of the national program of highway research and development.*

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PB 177754	Mineral Wear in Relation to Pavement Slipperiness.
PB 177758	Management Research Study of the Delaware State Highway Department.
PB 177759	Load Carrying Capacity of Minnesota Secondary Flexible Pavements.
PB 177782	Traffic Systems Reviews and Abstracts February Issue 1968.
PB 177813	Ultimate Strength of Rectangular Prestressed Concrete Columns Under Concentric and Eccentric Loadings.
PB 177815	Finite Element Analysis of Skew Composite Girder Bridges.
PB 177823	Multi-Directional Slip Base for Break-Away Luminaire Supports.
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PB 177831	An Analysis of Right-of-Way Appraisal Problems.
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PB 177890	Pile-Soil System Response in Clay Function of Excess Pore Water Pressure and Other Soil Properties.
PB [illegible]	Electronic Computer Study.
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PB 178292	The Effect of Flashing Traffic Control Devices on Accident Occurrence.
PB 178293	Investigation of Resistance of Concrete to Cracking.
PB 178294	Study of Lime Treated Sections on Road 10-SJ-580, 132.
PB 178295	Investigation of Dowel Bar Coatings.
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PB 178353	Insulation of Subgrade Third Year Summary
PB 178355	Analysis of Continuous Box Girder Bridges.
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PB 178357	Suitability of Synthetic Aggregates Made from Clay-Type Soils for Use in Flexible Base.
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Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1969). 35 cents.

Analysis and Modeling of Relationships between Accidents and the Geometric and Traffic Characteristics of the Interstate System (1969). $1.00.

The Bridge to Your Success (1969). 45 cents.

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Capacity Analysis Techniques for Design of Signalized Intersections (Reprint of August and October 1967 issues of PUBLIC ROADS, a Journal of Highway Research). 45 cents.

Construction Safety Requirements, Federal Highway Projects (1967). 50 cents.

Corrugated Metal Pipe Culverts (1966). 25 cents.

Creating, Organizing, & Reporting Highway Needs Studies (Highway Planning Technical Report No. 1) (1963). 15 cents.

Fatal and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967. 45 cents.

Federal-Aid Highway Map (42 x 65 inches) (1965). $1.50.

Federal Laws, Regulations, and Other Material Relating to Highways (1965). $1.50.

Federal Role in Highway Safety, House Document No. 93, 86th Cong., 1st sess. (1959). 60 cents.

Freeways to Urban Development, A new concept for joint development (1966). 15 cents.

Guidelines for Trip Generation Analysis (1967). 65 cents.

Handbook on Highway Safety Design and Operating Practices (Supplement No. 1, November 1968. 35 cents). 40 cents.

Highway Beautification Program. Senate Document No. 6, 90th Cong., 1st sess. (1967). 25 cents.

Highway Condemnation Law and Litigation in the United States (1968):
  Vol. 1—A Survey and Critique. 70 cents.
  Vol. 2—State by State Statistical Summary of Reported Highway Condemnation Cases from 1946 through 1961. $1.75

Highway Cost Allocation Study: Supplementary Report, House Document No. 124, 89th Cong., 1st sess. (1965). $1.00.

Highway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Research and Development Studies Using Federal-Aid Research and Planning Funds (1968). $1.50.

Highway Statistics (published annually since 1945):
  1965, $1.00; 1966, $1.25; 1967, $1.75.
  (Other years out of print.)

Highway Statistics, Summary to 1965 (1967). $1.25.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Human Values (Annual Report for Bureau of Public Roads) (1966). 75 cents.
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Identification of Rock Types (revised edition, 1960). 20 cents.
  Request from Bureau of Public Roads. Appendix, 70 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42, 89th Cong., 1st sess. (1965). 20 cents.

Interstate System Route and Log Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.
  Amendment No. 1 to above (1966). $1.00.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
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Modal Split—Documentation of Nine Methods for Estimating Transit Usage (1966). 70 cents.

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